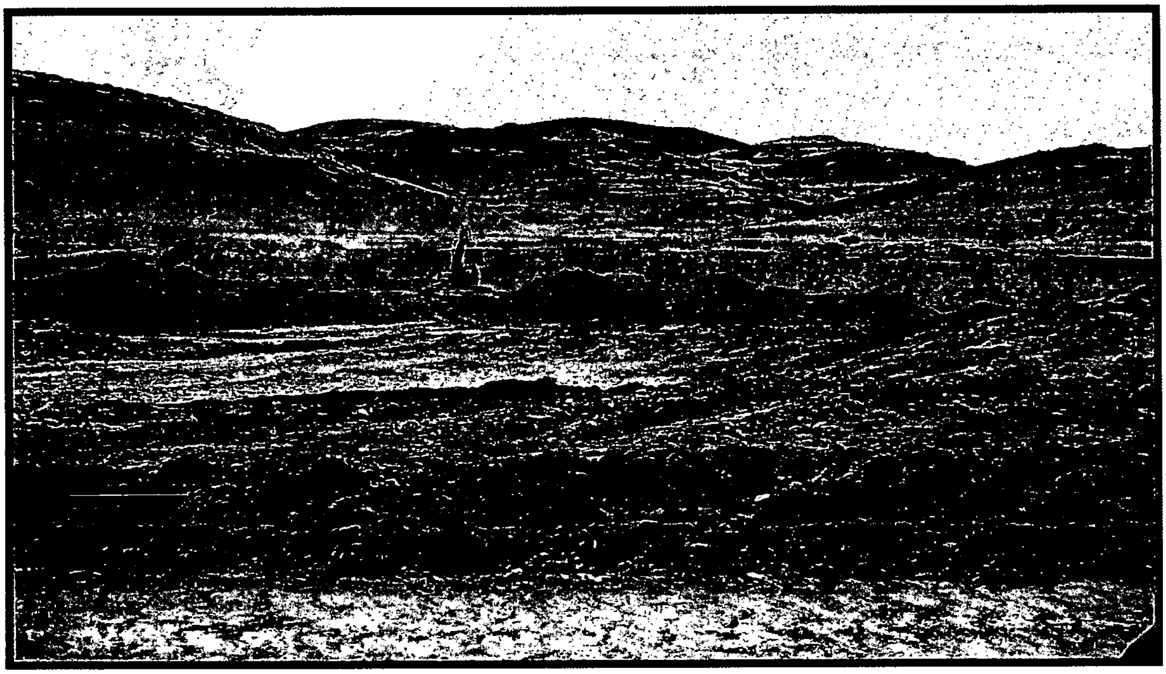
PERMIT MODIFICATION
for the
WASATCH REGIONAL LANDFILL
Tooele, Utah



Prepared for:

Allied Waste, Inc.
1111 West Highway 123
East Carbon, UT 84520
(435) 888-4418

express
RECEIVED
MAR 09 2009
UTAH DIVISION OF
SOLID & HAZARDOUS WASTE
2009.00888

Prepared by:

VECTOR
ENGINEERING, INC.
An **Ausenco** group company

143E Spring Hill Drive
Grass Valley, CA 95945
(530) 272-2448

Project No. 061204.11
February 2009

ECDC
ENVIRONMENTAL L.C.
AN ALLIED WASTE COMPANY

March 6, 2008

Dennis R. Downs, Director
Department of Environmental Quality
Division of Solid and Hazardous
288 North 1460 West
Salt Lake City, UT 84114-4880

Subject: Wasatch Regional Landfill, Inc. Request for Permit Modification, Landfill Height Increase, Addition of C&D Site, Alternative Fill Plan.

Dear Mr. Downs;

Please find enclosed an updated request Permit Modification document for Wasatch Regional Landfill, Inc. (WRL). WRL has included in the document an Alternative fill Plan, Attachment 3. The alternative plan is a stand-alone stability analysis document, which demonstrates the landfill is stable if construction of outside slope storm water controls benches, occurs after the slope is at final grade.

Currently, the storm water control benches are constructed in the waste mound during operation of the landfill. The Alternative Fill Plan will allow greater operation efficient. When a slope is at final grade, a contractor can be hired to construct the required control benches.

Vector Engineering prepared the report and completed the alternative fill stability analysis. The analysis did include the effects of the 100-foot vertical expansion. The report discusses the landfill is stable with the alternative fill method and minimal displacement occurs under earthquake loading.

The report also includes answers to questions previously requested by Division staff in a letter dated November 10, 2008.

Wasatch Regional understands a permit modification is required for approval of the vertical expansion and alternative fill concept and a new permit must be issued for the C&D landfill. Wasatch Regional thanks the DSHW in advance for a timely review and approval of the permit modification. If you have any questions please feel free to contact Jake Russell at 530-272-2448 or me at 435-888-4418 (22).

Sincerely,

Darin Olson
Republic Services, Mountain District
Environmental Manager

PERMIT MODIFICATION REQUEST
for the
WASATCH REGIONAL LANDFILL
Tooele, Utah

Prepared for:

Allied Waste, Inc.
1111 West Highway 123
East Carbon, UT 84520
(435) 888-4418

Prepared by:

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REGISTERED PROFESSIONAL ENGINEER
5881834-2202
JACOB V. RUSSELL
3/31/11
State of Utah

Project No. 061204.11
February 2009

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF ATTACHMENTS

1.0 INTRODUCTION

Allied Waste Industries, Inc. (Allied) is seeking to modify the configuration and operation of the Wasatch Regional Class V Landfill (WRL) by:

1. Increasing the maximum landfill elevation by approximately 100 feet.
2. Adding a Class VI, Construction and Demolition (C&D) cell within the existing landfill property.

This document describes the applicable features of the existing facility and the proposed modifications, and provides the engineering analyses performed in support of the modifications in compliance with the State of Utah Solid Waste Permitting and Management Rules R315-301 through 320.

2.0 LANDFILL DESCRIPTION

2.1 Location

The WRL is located west of the Great Salt Lake and adjacent to the east side of the Lakeside Mountain Range in Tooele County, Utah. The WRL is located west of Rowley Road in Tooele County, Utah, within Section 32, 33, and 34 of Township 2 North, Range 8 West, and within Sections 3 and 4 of Township 1 North, Range 8 West, Salt Lake Base and Meridian.

2.2 Climate

The site climate is arid with an average annual rainfall of 12.9 inches. Maximum precipitation months are March, April and May, whereas June, July and August are the drier months of the year. In addition, the site receives an average annual snowfall depth of 33.5 inches (Western Regional Climate Center).

2.3 Owner and Operator

The WRL is co-owned by Allied and the State of Utah School and Institutional Trust Lands Administration. It is operated by Allied.

2.4 Subsurface Conditions

The subsurface characteristics are described in Attachment 1 as part of the slope stability report.

2.5 Current Permit

The WRL currently operates under a permit issued by the Utah Division of Solid and Hazardous Waste. That permit was issued in association with the permit document titled "Municipal Solid Waste Landfill Permit Modification Design Engineering Report" (Hansen, Allen and Luce), dated December 2004 and revised in June 2005. The current permit does not include a provision for a Class VI cell at the landfill. It is the intent of this permit modification that the existing permit document remains in full effect relative to all WRL features and elements not addressed as part of this modification.

2.6 Current Landfill Configuration

The current configuration of the WRL is shown on Figure 1. The current ultimate configuration (master plan) for the WRL is shown on Figure 2. The final waste slopes are designed at 4H:1V with 25 foot-wide benches located every 50 feet vertically. The WRL was initially permitted for eleven phases covering approximately 793 acres with an ultimate gross airspace of approximately 160 million cubic yards.

The existing liner system consists of (from the bottom up):

- Prepared subgrade;
- Geosynthetic clay liner (GCL) (non-reinforced on the floor and reinforced on the sideslopes);
- 60-mil HDPE geomembrane (smooth on the bottom and textured on the sideslopes);
- Leachate collection and recovery system (LCRS) consisting of geonet overlain with non-woven geotextile filter fabric (on floor only); and
- Protective soil cover layer.

Existing stormwater control consists of a series of channels, benches, and downdrains which control run-on, from areas outside the landfill footprint and run-off, from areas within the landfill footprint. All stormwater from the site is diverted into the existing groundwater cutoff trench located to the east of the landfill. Stormwater controls are designed and constructed as the landfill expansion progresses.

3.0 FACILITY MODIFICATIONS

Two modifications are proposed for the WRL:

1. Increasing the maximum landfill elevation by approximately 100 feet, and
2. Adding a Class VI cell within the existing landfill property for construction and demolition (C&D) disposal.

This section describes the proposed modifications and presents the results of engineering analyses performed to support the modifications.

3.1 Vertical Expansion

The currently permitted maximum elevation of the WRL will be increased approximately 100 feet across the landfill footprint. This height increase will raise the maximum landfill elevation to approximately 4,620 feet. No associated horizontal expansion is proposed.

3.1.1 Configuration

The modified final cover grading plan is shown on Figure 3. The waste fill geometries (slopes, grades, benches) will remain the same as the current landfill. A typical section is shown on Figure 4. This modification will increase the gross landfill airspace from 160 million cubic yards to 220 million cubic yards.

The stability of the proposed configuration was analyzed using site specific soils and geosynthetic data obtained as part of project-specific laboratory testing programs performed for the last three expansions at the site. The methodology and results are presented in Attachment 1 titled *Waste Fill Stability Evaluation of the Wasatch Regional Landfill, Utah* (Vector 2008). The results of the stability analyses indicate that for static conditions the proposed landfill design is stable using the current liner system (FS = 1.7). The factor of safety for the pseudo-static condition was below 1.0 so a displacement analysis was performed. This analysis indicates displacements less than 1 inch for both liner options, which is also within acceptable industry standards for displacement during a seismic event. The static and seismic stability analysis and displacement analysis are discussed in detail in Attachment 1.

An infinite slope analysis was performed to check the stability of the final cover. Results and methods of this analysis are presented in detail in Attachment 1. The results of the analysis indicate the static factor of safety between 2.8 and 3.0 and pseudo-static factors of safety between 1.7 and 1.8.

3.1.2 Liner

The slope stability analyses performed were based on the current liner configuration. Based on the results of the stability analyses, the proposed landfill height increase will result in no changes to the liner system for the landfill.

3.1.3 Leachate Collection and Removal System

The proposed modification will require no changes to the leachate collection and removal system (LCRS) for the landfill.

The HELP model was run for the existing permit (Hansen, Allen, and Luce, 2004). The model was run for waste heights of 0, 10, 50, 100, and 200 feet. The results of the HELP modeling indicate that a waste height of 100 feet produces the highest peak daily discharge rate of 0.242 inches, and the annual leachate is the same for all heights. Based on this analysis and our experience with the HELP model, a vertical expansion of the landfill will reduce the peak daily leachate generation, therefore a recalculation of the leachate generation is not necessary for this permit modification. Performance of the geocomposite and leachate collection pipes under the additional loading was analyzed as described in the following sections.

3.1.3.1 Geonet / Geocomposite

The peak daily discharge rate of 0.242 inches from the HELP model was used for sizing the geonet in the existing permit for a 100' high waste height (Hanson, Allen, and Luce, 2004). At this rate the required transmissivity of the geocomposite was determined to be 1.023 x 10^{-3} m^2/sec. The requirement for a material that meets this transmissivity does not change for the additional waste thickness. However overburden loading, which has an effect on the transmissivity, will increase. In the current design documents, it was estimated that the overburden loading will range from 2,500 lb/ft^2 to 20,000 lb/ft^2 depending on the location within the landfill. Waste thickness generally increases in the landfill to the north and west with the maximum fill height occurring in the northwestern limits of the landfill. The additional waste will increase the maximum waste thickness to approximately 300 feet in this section, corresponding to a 22,500 lb/ft^2 overburden (assuming 75 lb/ft^3 as the unit weight of the waste as recommended by Kavazanjian (1999)). This increase in overburden pressure on the geocomposite will require the geocomposite be tested under higher loads during future design and construction projects. As in the existing permit, the required loading for geocomposite testing will be increased corresponding to the final waste thickness. According to GSE Lining Technology, Inc. a leading manufacturer of geocomposite material, products are available to provide the required transmissivity at the proposed loading.

The geocomposites previously installed in phases 1A, 1B, 2A, and 2B were evaluated for performance under the increased loading from the vertical expansion. The vertical expansion will increase the maximum depth of waste in parts of the existing landfill by approximately 75 feet for a maximum waste depth of 215 feet. Due to the gentle 4H:1V outer waste slopes, the majority of areas in the existing phases will remain unchanged and will have waste depths between 0 and 120 feet. Based on these waste depths, the maximum daily discharge rate from the HELP computer simulation results presented in the WRL Design Engineering Report by

Hansen Allen and Luce (2004) is 0.242 inch, corresponding to 100 feet of waste. The HELP simulation and past experiences indicate that increasing the height of waste will reduce the volume of daily leachate generated.

The McEnroe (McEnroe, 1993) and Giroud (Giroud et. al., 2000) methods for determining required transmissivity were used to re-evaluate the geocomposite transmissivity requirement to transport the daily leachate generated. Assuming a unit weight of 75 lb/ft^3 (Kavazanjian, 1999) for waste material, the maximum depth of approximately 215 feet corresponds to a maximum overburden pressure of 16,125 lb/ft^2 in the existing liner areas. Reduction factors were applied to account for degradation of the geocomposite throughout the life of the landfill (GRI GC8, 2001). Table 1 shows the input parameters used in the McEnroe and Giroud equations.

TABLE 1
TRANSMISSIVITY CALCULATION PARAMETERS

PARAMETER	DEFINITION	VALUE
S	Slope of landfill floor	2.68%
Qh	Inflow (from HELP)	0.242 in/day
L	Length of leachate flow in geocomposite	140 ft
tLCL	Thickness of LCRS layer	2 ft
RFin	Intrusive Reduction Factor	1.2
RFcr	Creep Reduction Factor	3.5
RFcc	Chemical Clogging Reduction Factor	1.5
RFBC	Biological Clogging Reduction Factor	1.3
FSd	Overall factor of safety for drainage	2

The creep reduction factor, RFcr, is influenced by the compressibility of the geocomposite core and is intended to account for the reduction in cross-sectional area that occurs under large, sustained loading. The creep reduction factor can be determined from laboratory strain tests on the geocomposite core. Typical strain tests (such as ASTM D6364) are time consuming tests that can take longer than 10,000 hours to conduct (ASTM D6364, 2004). As an alternative, a conservatively high creep reduction factor of 3.5 was assumed in the analysis. The typical range for creep reduction factors is from 1.4 to 2.0 (Koerner, 1994). Furthermore, the GSE Fabrinet HF, installed in phases 2A and 1B can be expected to creep approximately 50% (RFcr = 1.5) under a 25,000 lb/ft^2 loading based on previously conducted research (Li, 2008). Therefore, the 3.5 creep reduction factor used in the analyses is conservative for the loads resulting from the height increase.

Based on the analysis performed for the existing geocomposites and the proposed overburden, the existing landfill phases will require a geocomposite with a transmissivity of 1.02x10-3 m2/s based on the McEnroe solution or 1.80x10-3 m2/s based on the Giroud solution. The McEnroe and Giroud calculation sheets are shown in Attachment 2. The project specifications for the LCRS geocomposites used in the four existing landfill phases are listed in Table 2. In all previously constructed phases, the project specifications are greater than the minimum required transmissivity determined from the McEnroe and the Giroud solutions.

TABLE 2
SUMMARY OF INSTALLED GEOCOMPOSITES

PHASE	GEONET/GEOCOMPOSITE IN PLACE	PROJECT TRANSMISSIVITY SPECIFICATION (M2/SEC) ASTM D 4716
1A	200 mil HyperNet (XL4000N004)	1x10-3 @ 12,000 psf
1B	GSE Fabrinet HF XL5 (F510800005)	1x10-3 @ 12,000 psf
2A	GSE Fabrinet HF XL5 (F510800005)	1x10-3 @ 12,000 psf
2B	Skaps TN220-1-8	1x10-3 @ 12,000 psf

Third party geosynthetics conformance testing conducted during construction verified that the geocomposites installed in each phase met or exceeded the project specifications.

Based on the results of the above analysis the geocomposites currently installed in the existing phases of the landfill will perform as designed under the increased loading from the vertical expansion.

3.1.3.2 Leachate Collection Pipe

The 8" ADS Type C CPE leachate collection pipes currently used for leachate collection and transport to the sumps were evaluated for excessive deflection from the increased overburden pressure using the Burns-Richard solution. The Burns-Richard solution is an empirical method of estimating pipe deflections based on field and laboratory observations which uses pipe and surrounding soil material properties to determine pipe reaction to overburden.

The Burns-Richard Solution for the ADS 8" corrugated pipe currently installed at WRL estimated pipe deflections from the overburden to be approximately 7%, or 0.6 inch. This calculation shows that under the maximum overburden pressure the pipe used for the LCRS will be structurally sound and the additional pressure will not cause significant deformation. Pipe deflection calculations are included in Attachment 2. The 100 ft. vertical expansion will not warrant additional engineering or design changes for piping used for the LCRS. Additionally pipes currently installed in existing phases of the landfill will perform as designed under the additional loading from the vertical expansion.

3.1.4 Stormwater Control

The proposed modification will result in no changes to the overall drainage area or site hydrology. The existing stormwater control facilities and drainage flow patterns will, at a conceptual level, remain the same. Detailed design for the drainage control facilities will be conducted as build-out of the landfill progresses taking into account the revised final configuration of the landfill.

3.1.5 Monitoring Facilities

The proposed modification will result in no changes to the existing monitoring facilities.

3.2 Class VI Cell

A new, hydraulically-separated cell will be constructed adjacent to the existing landfill for the disposal of construction and demolition material. The cell will be permitted as a Class VI cell in accordance with the State of Utah Solid Waste Permitting and Management Rules R315-301-2(12). The Class VI cell is adjacent to the existing landfill and thus the site characteristics associated with the new cell are consistent with those for the landfill.

3.2.1 Configuration

The Class VI cell bottom grading plan is shown on Figure 5. The sideslopes will be graded at 2H:1V and the bottom will be graded flat. The maximum depth of the excavation will be approximately 34 feet. The final grading plan is shown on Figure 6. The maximum height of the fill will be approximately 100 feet, with 3H:1V slopes and no intermediate benches and a top deck slope of 5%. The cell will have a footprint area of approximately 488,000 square feet (11.2 acres) and an estimated gross capacity of 780,000 cubic yards. A 30 foot wide perimeter road will be designed around the Class VI cell and between the Class VI cell and the existing Class V landfill.

3.2.2 Liner

The Class VI cell will be unlined.

3.2.3 Leachate Collection and Removal System

The Class VI cell will not have a leachate collection and removal system.

3.2.4 Stormwater Control

Drainage and collection structures for surface runoff will be designed to contain a 25-year storm. The design will also include elements to prevent surface water run-on from a 25-year storm.

3.2.5 Final Cover

The Class VI cell will use the evapotranspirative final cover described in the report entitled *Evapotranspirative (ET) Final Cover Permitting Report for the Wasatch Regional Landfill*, Vector Engineering, June 2004.

4.0 REFERENCES

ASTM D6364, "Standard Test Method for Determining Short-Term Compression Behavior of Geosynthetics", 2004

Bray, J., Rathje, E., Augello, A., and Merry, S. (1998) Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills, Geosynthetics International, Vol. V(1-2), pp. 204-235.

Giroud, J.P., Zornberg, J.G., and Zhao, A., 2000, "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers", *Geosynthetics International*, Special Issue on Liquid Collection Layers, Vol. 7, Nos. 4-6.

GRI, "Determination of the Allowable Flow Rate of a Drainage Geocomposite", GRI Standard – GC8, 2001

Hansen, Allen & Luce, Inc. Consulting Engineers. (2004). Wasatch Regional Landfill – Municipal Solid Waste Landfill Permit Modification Design Engineering Report. December, 2004.

Kavazanjian, E. (1999), Seismic design of solid waste containment facilities, Eighth Canadian Conference on Earthquake Engineering, 13 – 15 June, 1999.

Koerner, R. M., "Designing with Geosynthetics", Third Edition, Prentice Hall, 1994.

Li, Mengjia. "RE: GSE FabriNet HF XL5." E-mail to Mengijia Li, Drainage and Geotextile Product Manager, GSE Lining Technologies Inc. 24 Nov. 2008.

McEnroe, B., "Maximum Saturated depth over Landfill Liners", Journal of Environmental Engineering, Vol. 19, No. 2, March/April, 1993.

Seed, H. B., 1979, "Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams", 19th Rankine Lecture of British Geotechnical Society, Geotechnique 29, No. 3, p. 215-263, London, England.

Vector Engineering, Inc. (2004). Evapotranspirative (ET) Final Cover Permitting Report for the Wasatch Regional Landfill. June 2006.

Vector Engineering, Inc. (2008). Waste Fill Stability Evaluation of the Wasatch Regional Landfill, Utah. July 2008.

FIGURES

LEGEND

	EXISTING 10 FT CONTOUR
	EXISTING 2 FT CONTOUR
	EXISTING 25 FT CONTOUR
	EXISTING 5 FT CONTOUR
	EXTENTS OF PHASE

NOTES:
1. EXISTING TOPOGRAPHY BASED ON AERIAL SURVEY PERFORMED BY OLYMPUS AERIAL SURVEYS, INC. ON MARCH 3, 2008

NO.	DATE	DESCRIPTION	DRAWN BY	DESIGNED BY	CHECKED BY	APPROVED BY

DATE OF ISSUE: 07/30/2008
DESIGNED BY: BGA
DRAWN BY: RPB
CHECKED BY: JVR
APPROVED BY: JVR

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An Ausenco Group Company
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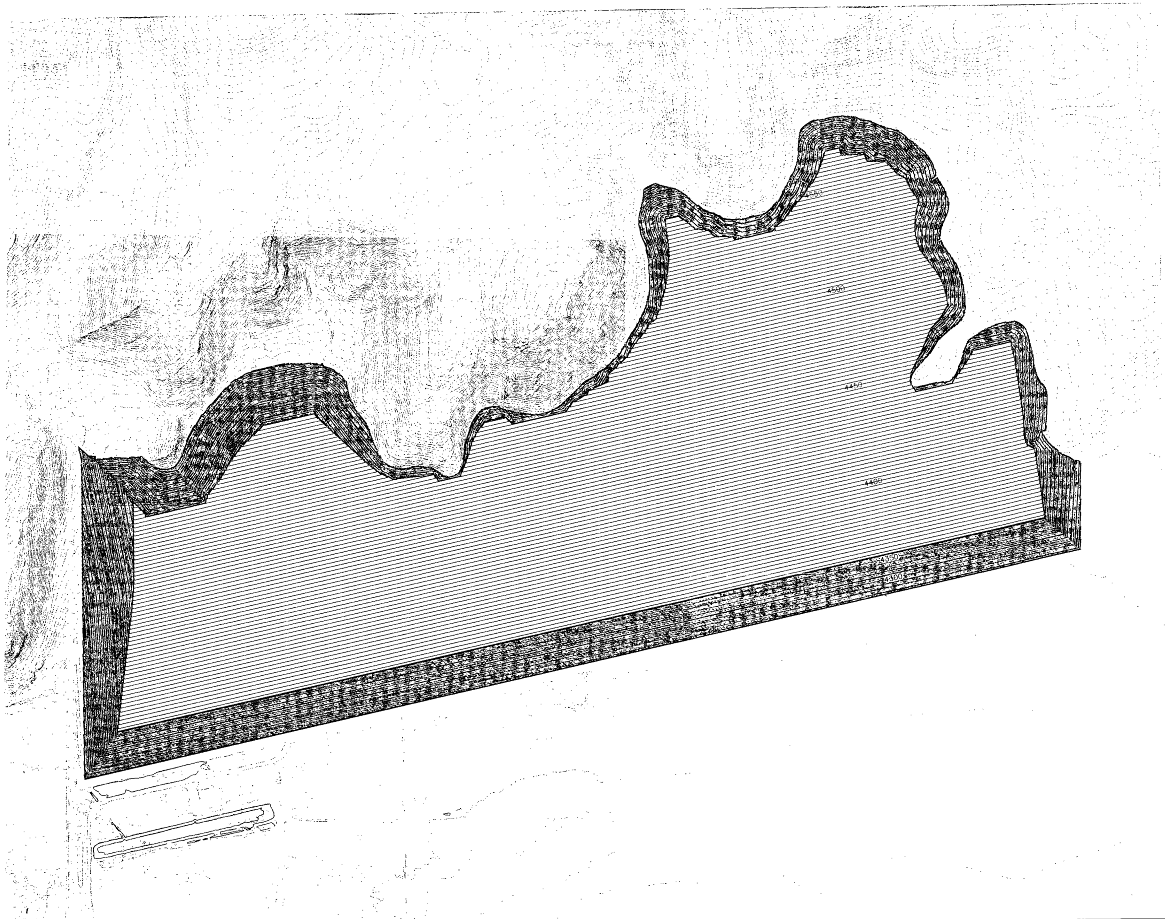
WASATCH REGIONAL LANDFILL, INC.

WASATCH REGIONAL LANDFILL
PERMIT REVISION
TOOELE COUNTY, UTAH
EXISTING SITE CONDITIONS

FIGURE NO. 1
PROJECT NO. 061204.11

PERMIT DRAWING



GRAPHIC SCALE

500' 0' 250' 500' 1000' 2000'

LEGEND

	EXISTING 10 FT CONTOUR
	EXISTING 2 FT CONTOUR
	EXISTING 25 FT CONTOUR
	EXISTING 5 FT CONTOUR
2860	EXISTING PERMIT 10 FT CONTOUR (2)
	EXISTING PERMIT 2 FT CONTOUR (2)

QUANTITIES

EXISTING PERMIT AIRSPACE = 160,000,000 CY

NOTES:

1. EXISTING TOPOGRAPHY BASED ON AERIAL SURVEY PERFORMED BY OLYMPUS AERIAL SURVEYS, INC. ON MARCH 3, 2008.
2. BENCHES WERE INCLUDED IN DESIGN OF EXISTING FINAL COVER BUT NOT SHOWN IN GRADING PLAN.

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DATE OF ISSUE: 07/30/2008
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APPROVED BY: JVR

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WASATCH REGIONAL LANDFILL, INC.

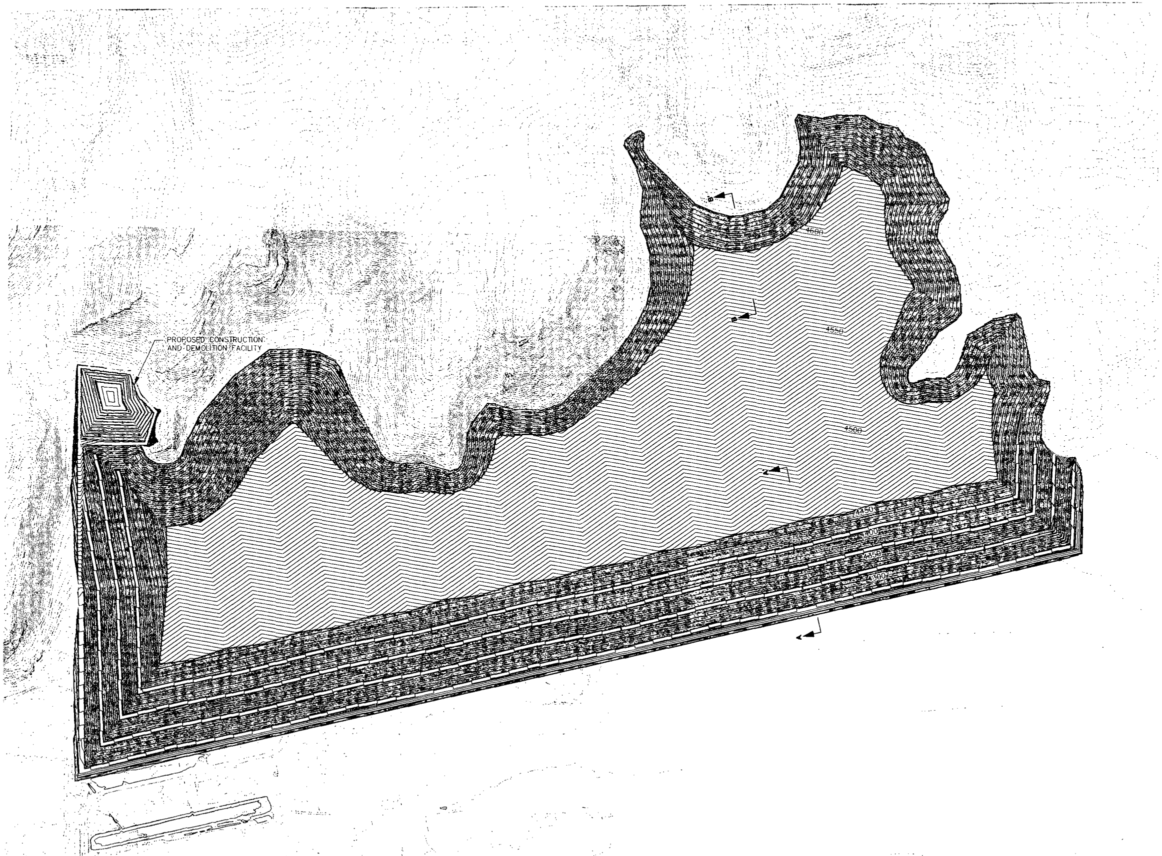
WASATCH REGIONAL LANDFILL
PERMIT REVISION
TOOELE COUNTY, UTAH

EXISTING PERMIT FINAL COVER GRADE

FIGURE NO. 2

PROJECT NO. 061204.11

PERMIT DRAWING



GRAPHIC SCALE

500' 0' 250' 500' 1000' 2000'

LEGEND

	EXISTING 10 FT CONTOUR
	EXISTING 2 FT CONTOUR
	EXISTING 25 FT CONTOUR
	EXISTING 5 FT CONTOUR
2860	PROPOSED FINAL COVER 10 FT CONTOUR (2)
	PROPOSED FINAL COVER 2 FT CONTOUR (2)
	C&D FACILITY LIMITS

QUANTITIES

REVISED FINAL FILL PLAN AIRSPACE = 220,000,000 CY

NOTES:
1. EXISTING TOPOGRAPHY BASED ON AERIAL SURVEY PERFORMED BY OLYMPUS AERIAL SURVEYS, INC. ON MARCH 3, 2008
2. FUTURE BENCHES NOT SHOWN ON WEST SIDE OF LANDFILL

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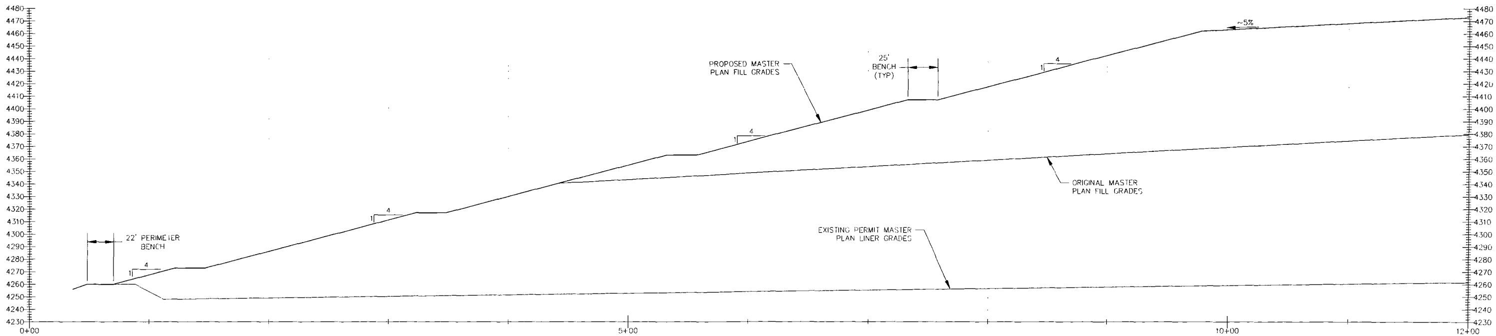
WASATCH REGIONAL LANDFILL, INC.

WASATCH REGIONAL LANDFILL
PERMIT REVISION
TOOELE COUNTY, UTAH
LANDFILL REVISED FINAL COVER GRADE

FIGURE NO. 3
PROJECT NO. 061204.11

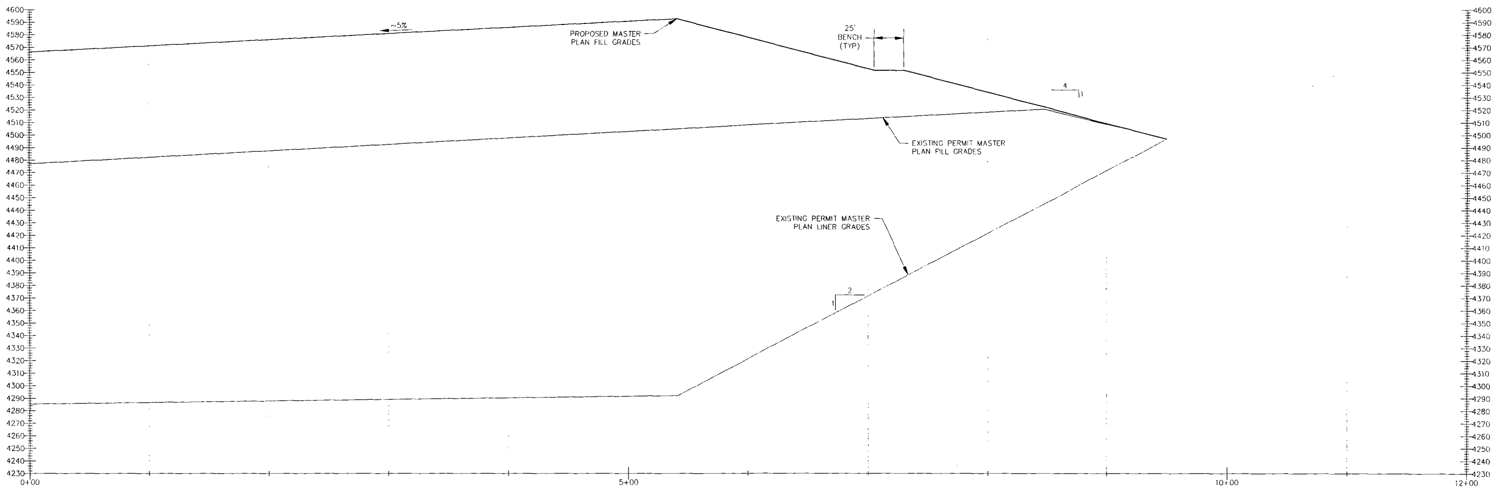
PERMIT DRAWING



SECTION A-A'

1" = 40'

B03 | B04



SECTION B-B'

1" = 40'

B03 | B04

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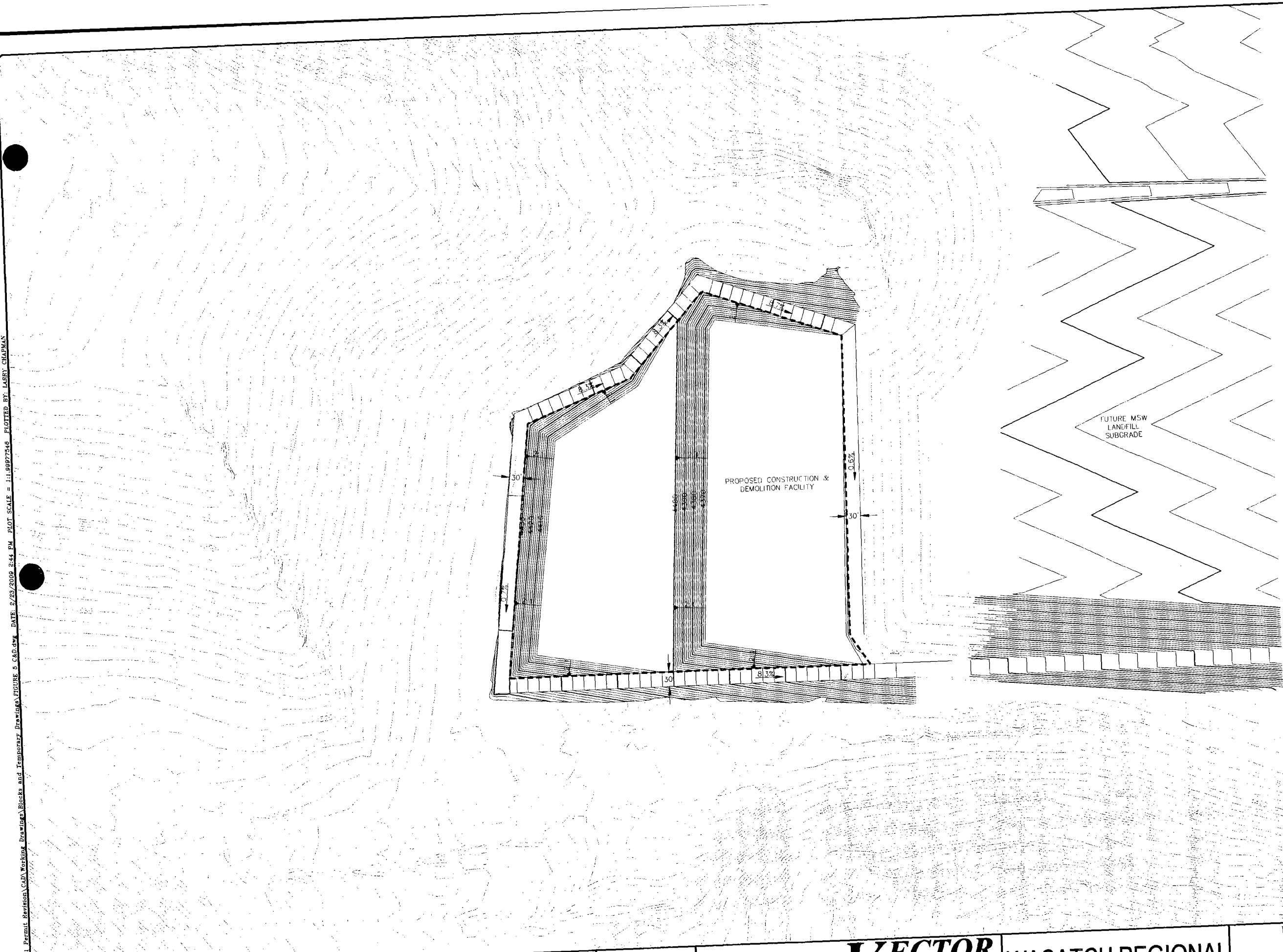
WASATCH REGIONAL LANDFILL, INC.

WASATCH REGIONAL LANDFILL
PERMIT REVISION
TOOELE COUNTY, UTAH
CROSS SECTIONS

FIGURE NO. 4
PROJECT NO. 061204.11

PERMIT DRAWING



GRAPHIC SCALE

100' 0' 50' 100' 200' 400'

LEGEND

- EXISTING 10 FT CONTOUR
- EXISTING 2 FT CONTOUR
- 2860 FUTURE 10 FT SUBGRADE CONTOUR
- FUTURE 2 FT SUBGRADE CONTOUR
- —x—x— EXISTING FENCE
- C&D FACILITY LIMITS

QUANTITIES

C&D FACILITY
EXCAVATION = 294,508 CY
ENGINEERED FILL = 8,662 CY
FACILITY FLOOR AREA = 487,500 SF

NOTES:
1. EXISTING TOPOGRAPHY BASED ON AERIAL SURVEY PERFORMED BY OLYMPUS AERIAL SURVEYS, INC. ON MARCH 3, 2008

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DRAWN BY: RPB
CHECKED BY: JVR
APPROVED BY: JVR

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WASATCH REGIONAL LANDFILL, INC.

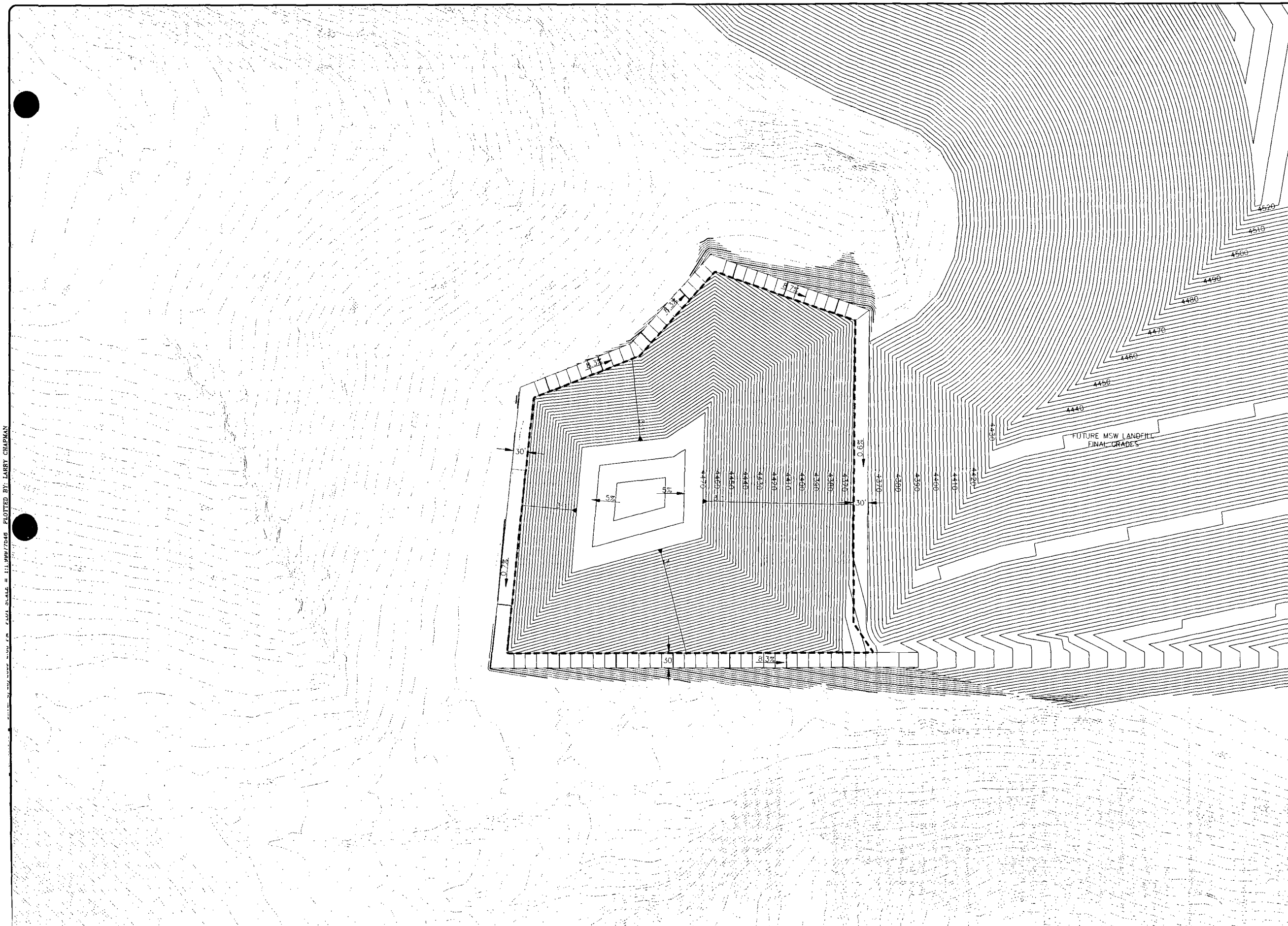
WASATCH REGIONAL LANDFILL
PERMIT REVISION
TOOELE COUNTY, UTAH
C&D FACILITY SUBGRADE PLAN

FIGURE NO. 5
PROJECT NO. 061204.11

PERMIT DRAWING

This drawing has not been published but rather has been prepared by Vector Engineering, Inc. for use by the client named in the title block, solely in respect of the construction operation, and maintenance of the facility named in the title block. Vector Engineering, Inc. shall not be liable for the use of this drawing on any other facility or for any other purpose.

LOCATION: N:\Wasatch\061204.11 Permit Revision\CAD\Working Drawings\Blocks and Temporary Drawings\FIGURE 5 C&D.dwg DATE: 2/23/2009 2:44 PM PLOT SCALE = 1:1.99977546 PLOTTED BY: LARRY CHAPMAN



GRAPHIC SCALE

100' 0' 50' 100' 200' 400'

LEGEND

	EXISTING 10 FT CONTOUR
	EXISTING 2 FT CONTOUR
2860	FUTURE FINAL 10 FT COVER CONTOUR
	FUTURE FINAL 2 FT COVER CONTOUR
2860	FUTURE 10 FT C&D ROAD GRADING CONTOUR
	FUTURE 2 FT C&D ROAD GRADING CONTOUR
—x—x—	EXISTING FENCE
------	C&D FACILITY LIMITS

QUANTITIES

C&D FACILITY
CAPACITY = 779,524 CY
COVER SURFACE AREA = 464,870 SF
COVER SOIL VOLUME = 45,371 CY[2]

NOTES:
1. EXISTING TOPOGRAPHY BASED ON AERIAL SURVEY PERFORMED BY OLYMPUS AERIAL SURVEYS, INC. ON MARCH 3, 2008
2. BASED ON 2.5' OF PROTECTIVE SOIL COVER.

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WASATCH REGIONAL LANDFILL, INC.

WASATCH REGIONAL LANDFILL
PERMIT REVISION
TOOELE COUNTY, UTAH
C&D FACILITY FINAL COVER GRADE

FIGURE NO. 6
PROJECT NO. 061204.11

PERMIT DRAWING

ATTACHMENT 1
WASTE FILL STABILITY EVALUATION OF THE WASATCH LANDFILL, UTAH. JULY 2008

WASTE FILL STABILITY EVALUATION
of the
WASATCH REGIONAL LANDFILL
Toole County, Utah

Prepared for:
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TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

1.1 Purpose

The purpose of this analysis was to evaluate the slope stability for a 100-ft increase in maximum waste height at the Wasatch Regional Landfill (WRL), located in Tooele County, Utah. The stability evaluation was performed by Vector Engineering, Inc. (Vector), and is summarized in this report.

1.2 Scope of Work

Vector's scope of work included conducting a soils investigation in 2006 and an evaluation of the final liner system options and waste fill configurations for the WRL. Slope stability analyses were performed to ensure the static and pseudo-static stability of the system, and included the following critical design elements:

1. An increase in the top deck elevation of the landfill by 100 feet, which would raise the maximum waste elevation to 4,620 feet.
2. A maximum overall waste slope of 4 horizontal to 1 vertical (4H:1V), with a top deck slope of approximately 5%.
3. Side slopes lined with textured geomembrane and high-strength geosynthetic clay liner (GCL).
4. A floor-liner system comprised of GCL, either smooth or textured geomembrane, and a geocomposite.

The work tasks performed for this study included the following:

1. *Laboratory Testing.* Large Scale Direct Shear (LSDS) tests for several liner system configurations were performed in October 2006, May 2007, and April 2008. All laboratory testing was conducted by Vector in Grass Valley, California.
2. *Seismic Hazard Evaluation.* Historic, deterministic, and probabilistic analyses were performed to evaluate the site specific seismic risks and potential slope stability hazards.
3. *Slope Stability Analyses.* Limit-equilibrium slope stability analyses were performed for an idealized cross section of the landfill. Infinite Slope stability analyses were performed on the final cover system. Slope stability was evaluated for static and pseudo-static (earthquake) conditions.

4. *Displacement Analyses.* Based on the results of the pseudo-static stability analyses, potential displacements were estimated for the design earthquake magnitude.
5. *Report Preparation.* This report summarizes the results and conclusions for each of the tasks listed above.

1.3 Location and General Description

The WRL is located at 8833 North Rowley Road, North Skull Valley, Utah; west of the Great Salt Lake and adjacent to the east side of the Lakeside Mountain Range in Tooele County. The WRL will consist of eleven phases covering approximately 793 acres and will have an ultimate capacity of approximately 160 million cubic yards.

The site climate is arid with an average annual rainfall of 12.9 inches. Maximum precipitation months are March, April and May, whereas June, July and August are the drier season. In addition, the site receives an average annual snowfall depth of 33.5 inches (Western Regional Climate Center).

In the final configuration, the waste slopes will be graded at a maximum slope of 4H:1V in between benches, with a top deck slope of approximately 5 percent. The slope will have benches that are approximately 25 feet wide. The highest slope is located on the east side of the landfill running in a north-south direction, having a vertical slope height of approximately 200 ft. The expansion will have a liner and a leachate collection system as well, and therefore, a leachate mound is not expected to develop within the landfill and was not included in the analyses. The critical landfill cross-sections used for the stability analyses are shown in Appendix D.

The side-slope liner system consists of the following elements (from bottom to top):

- Prepared subgrade;
- Reinforced GCL installed over the prepared subgrade;

- 60-mil double textured HDPE geomembrane covering the GCL; and
- A 2-ft thick layer of protective soil cover.

Two different options for the floor liner system were analyzed. The elements of floor liner system OPTION 1 included (from bottom to top):

- Prepared subgrade;
- Non-reinforced GCL installed over the prepared subgrade;
- 60-mil smooth HDPE geomembrane covering the GCL;
- Single sided geocomposite drainage layer over the geomembrane; and
- A 2-ft thick layer of protective soil cover.

The elements of floor liner system OPTION 2 included (from bottom to top):

- Non-reinforced GCL installed over the prepared subgrade;
- 60-mil double sided textured HDPE geomembrane covering the GCL;
- Single sided geocomposite drainage layer over the geomembrane; and
- A 2-ft thick layer of protective soil cover.

2.0 SUBSURFACE INVESTIGATION AND CONDITIONS

2.1 Field Investigation

Previous geotechnical investigations for the WRL were conducted by AGEC (2004, 2005) and Kleinfelder (2004). In addition, Vector conducted logging and sampling of four soils from test pits excavated in 2006. Classification tests were performed for the samples, including initial moisture (ASTM D-2216), particle size analysis (ASTM D-422), and Atterberg limits (ASTM D-4318).

2.2 Laboratory Testing

For the purpose of this study, additional laboratory testing was performed by Vector in April 2008. LSDS tests were completed to obtain shear strength properties for the following interfaces: GCL vs. Double Textured HDPE, GCL vs. Smooth HDPE, Single Sided Textile Geocomposite vs. Smooth HDPE, GCL vs. GCL and Double Textured HDPE vs. GCL. All of the laboratory test results are presented in Appendix A.

2.3 Subsurface Conditions

Subsurface information presented within this report was obtained from the Geotechnical Investigation Permit Modification prepared by AGEC (2004) for the WRL. Subsurface conditions at the site were characterized by exploratory borings drilled by AGEC and the subsurface information reported by Kleinfelder and Vector. The subsurface profile generally consists of clay, silt and fine sand on the lower elevation portions of the site, with coarser grained materials present at higher elevations. Limestone bedrock was encountered in boring B-1 (AGEC, Dec. 2004) at a depth of 143 ft. Boring B-1 is located at local coordinates North 7,479,138.81 and East 1,293,915.65 (AGEC, Dec. 2004). The clay at the site is interlayered with sandy silt and occasionally silty sand. The clay is stiff to very stiff, slightly moist to moist, and brownish gray in color. The silty clay is gray in color, and medium stiff to soft. The silty sand contains occasional lean clay layers and ranges from loose to dense. The sandy gravel is silty and clayey, but contains occasional cobbles and boulders, and ranges from medium to very dense.

3.0 GEOLOGIC SETTING AND SITE GEOLOGY

3.1 Geologic Setting

The WRL is located in the Basin and Range Geomorphic province, which is characterized by horst and graben structure (subparallel, fault-bounded ranges separated by downdropped basins). This portion of the Basin and Range is within the Great Basin province, characterized by interior drainage with lakes and playas. The Basin and Range began extension during the Miocene. Many of the ranges are bounded by high-angle normal faults.

The exposed bedrock within the ranges in this portion of the Great Basin is predominantly Precambrian and Paleozoic marine carbonate and clastic sedimentary rocks (limestone, dolomite, shale, sandstones) with subordinate amounts of Tertiary volcanics. The intervening valleys within the Basin and Range are composed of alluvial, lacustrine and volcanic materials as much as 8,000 feet thick that have been deposited more-or-less continuously since the Miocene (within the last 15 million years).

During Late Pleistocene time, Lake Bonneville formed in western Utah and reached its highest level approximately 15,000 to 20,000 years ago. Lake Bonneville reached a maximum depth of over 1,000 feet, which resulted in many of the ranges in the area becoming islands. Since that time, Lake Bonneville has been shrinking to the size of the Great Salt Lake.

3.2 Site Geology

The WRL is located on the eastern edge of the Lakeside Mountains. These mountains are oriented north-south and are a northern extension of the Cedar Mountains. The Great Salt Lake shoreline is approximately 2.5 miles east of the Site. According to Hintze et al. (2000), the site is underlain by lacustrine sediments that were deposited during the Late Pleistocene when the surface of Lake Bonneville was about 900 feet above the site.

The Lakeside Mountains west of the site are composed of Paleozoic marine sedimentary rocks folded into a syncline plunging to the southeast. The core of the syncline contains Mississippian aged Woodman Formation and/or Ochre Mountain Limestone with the northern limb of the syncline containing Ordovician through Devonian age dolomites, limestones, shales and sandstones. The outcrops immediately west of the site are part of the Devonian section. The southern limb of the syncline has been largely faulted away, with Pennsylvanian to Permian rocks exposed on the south side of the fault.

Below the lacustrine sediments that underlie the site, bedrock is likely to exist at a relatively shallow depth along a peneplane as evidenced by small presumably bedrock knobs east of the site.

4.0 FAULTING, SEISMOLOGY & EARTHQUAKE GROUND MOTION

Deterministic seismic hazard analyses were conducted for 12 fault sources within a 160 km radius of the WRL to provide the potential ground motion seismic evaluation of the waste fill stability.

4.1 Local and Regional Faulting

The WRL is located approximately 72 km west from the Wasatch Front area, which is a seismically active region having only moderate historical seismicity, but high catastrophic potential from future large earthquakes. The Wasatch Fault is one of the longest and most tectonically active normal faults in North America which slips in a primarily vertical direction, with the mountains rising relative to the valley floor. The fault zone shows abundant evidence of recurrent Holocene surface faulting and has been the subject of detailed studies for over three decades. This fault has 10 sections where the southern 8 sections are entirely in Utah. The nearly 350-km-long Wasatch fault zone has traditionally been divided into seismogenic segments that are thought to behave at least somewhat independently. The chronology of surface-faulting earthquakes on the fault is one of the better dated in the world, and includes 16 earthquakes within the last 5,600 years, with an average repeat time of 350 years. Four of the central five sections ruptured between 600 and 1,250 years ago; whereas the next section to the north has not ruptured in the past 2,125 years. Slip rates of 1-2 mm/yr are typical for the central sections during Holocene time. In contrast, middle and late Quaternary (<150-250 ka) slip rates on these sections are about an order of magnitude lower.

The closest fault which U.S. Geological Survey (USGS) indicates as active during the Latest Quaternary (within the last 15,000 years) is the west side of Stansbury Fault which is located approximately 14 km south of the site. The Stansbury Fault is located along the western side of the Stansbury Mountains. This is a generally north-trending normal fault zone bounding the western side of the Stansbury

Mountains. The Stansbury Mountains expose mainly Paleozoic rock, and are the centermost of three prominent north-south mountain ranges (including the Oquirrh Mountains to the east and Cedar Mountains to the west) west of the high central part of the Wasatch Range. Surficial geology in the valleys between the ranges is dominated by lake deposits and alluvium. The USGS describes the Stansbury Fault as a normal fault with latest activity occurring in Holocene to Late Quaternary time with a slip rate of less than approximately 0.2 mm/yr.

4.2 Historical Seismicity

As early as 1883, geologists recognized and warned of the serious earthquake threat posed by the Wasatch Fault and other active faults in Utah despite the absence, up to that time, of any large earthquakes in the region. A search of historical earthquakes occurring between 1800 and 2008, listed in the USGS catalog, was performed for a 160 km radius around the project site. That search found that 605 earthquakes occurred within that area during that 208-year period. Of those earthquakes, 11 have moment magnitudes (Mw) of 5 or greater, and 3 have Mw of 6 to 6.8.

The largest recorded near-source earthquake to affect the area within a 160 km radius was an Mw 6.8 that occurred on March 12, 1934, approximately 74 km from the project site. According to USGS, the closest historical earthquake to affect the site was an Mw 5.2 event that occurred approximately 35 km east of the site. The largest estimated site acceleration to affect the area within a 160 km radius occurred on March 12, 1934 and March 28, 1975. These events were located approximately 74 km and 135 km, respectively, from the project site. Table 1 summarizes the peak horizontal acceleration of the mentioned historical earthquakes at the site, according to various attenuation relationships.

TABLE 1
SUMMARY OF PEAK HORIZONTAL ACCELERATIONS FOR HISTORICAL EARTHQUAKES

EARTHQUAKE MAGNITUDE (Mw)	DATE OF EVENT	DISTANCE FROM SITE (km)	PEAK HORIZONTAL ACCELERATION (G)			
			BOORE ET AL. (1993)	TORO ET AL. (1995)	YOUNGS ET AL. (1988)	AVERAGE PEAK HORIZONTAL ACCELERATION (g)
5.2	Sept. 5, 1962	35	0.030	0.050	0.03	0.037
6.8	March 12,1934	74	0.079	0.100	0.12	0.100
5.1	March 12, 1934	74	0.000	0.000	0.01	0.000
6.1	March 12, 1934	74	0.040	0.075	0.03	0.048
5.3	April 14, 1934	74	0.000	0.000	0.00	0.000
5.5	May 6, 1934	74	0.040	0.070	0.05	0.053
5	May 24, 1980	120	0.000	0.000	0.00	0.000
5.5	April 7, 1934	127	0.010	0.100	0.03	0.047
6.8	March 28, 1975	135	0.045	0.100	0.06	0.068
5.7	Aug. 30, 1962	157	0.01	0.06	0.01	0.036

4.3 Deterministic Estimates of Strong Ground Motions

Peak horizontal ground accelerations were estimated for the project site using the attenuation relationship from Idriss (1991). A search was conducted for all earthquake sources within a 160 km radius of the project site which are believed to be active during Holocene time (the last 10,000 years). The activity and location of the faults was based on information from the USGS. From this search, it was determined that there are 72 earthquake sources which are believed to be active within a 100-mile radius of the site. The results of the deterministic estimates for the 12 earthquakes with the highest estimated Peak Ground Acceleration (PGA) are shown in Table 2. A more comprehensive list of earthquake sources is presented in Appendix B.

TABLE 2
DETERMINISTIC GROUND MOTION DATA

FAULT NAME	UPPER BOUND EARTHQUAKE (M_W)	DISTANCE FROM SITE (km)	APPROXIMATE FAULT DATA		DETERMINISTICALLY ESTIMATED PEAK GROUND ACCELERATION (G)
			LENGTH (Km)	SLIP RATE (MM/YR)[A]	M [B]
Stansbury fault zone	6.9	14	50	less than 0.2	0.436
Skull Valley (mid-valley) faults	6.9	35	55	less than 0.2	0.182
Puddle Valley fault zone	6.1	24	7	less than 0.2	0.136
Oquirrh fault zone	7.0	47	21	0.2 to 1	0.135
East Great Salt Lake fault zone, Promontory section	6.8	48	37	0.2 to 1	0.121
East Great Salt Lake fault zone, Antelope Island section	6.6	40	26	0.2 to 1	0.110
Southern Oquirrh Mountains fault zone	7.1	58	24	0.2 to 1	0.109
East Great Salt Lake fault zone, Fremont Island section	6.3	40	13	0.2 to 1	0.086
Wasatch fault zone, Salt Lake City section	7.1	72	23	1 to 5	0.083
Wasatch fault zone, Weber section	7.0	72	20	1 to 5	0.079
Wasatch fault zone, Clarkston Mountain section	7.3	80	43	less than 0.2	0.079
Wasatch fault zone, Provo section	7.1	80	23	1 to 5	0.072

[A] From USGS
[B] *M = indicates* estimated mean peak horizontal ground acceleration from Idriss (1991).

Based on these evaluations, the site could be subjected to horizontal ground accelerations as high as 0.436 g from the rupture along the Stansbury Fault. The Stansbury Fault zone is located about 14 km south of the site. It should be noted that probability and exposure periods are not considered during deterministic evaluations and that, typically, deterministic estimates of strong ground motion for a site generate relatively conservative horizontal ground acceleration values.

4.4 Probabilistic Estimates of Strong Ground Motion and Peak Ground Acceleration

Probabilistic evaluations of horizontal ground motions that could affect the site were performed using the USGS *Java Ground Motion Parameter Calculator – Version 5.0.8*. This application includes hazard curves, uniform hazard response spectra, and design parameters for sites in the 50 states of the United States, Puerto Rico, and the U.S. Virgin Islands. Parameters were searchable with the latitude and longitude data of the WRL, which are approximately 40.85 latitude and -112.75 longitude. The application was used to obtain uniform hazard response spectra for 2% probability of exceedance in 50 years and 10% probability of exceedance in 50 years. Table 3 summarizes the probabilistic ground motion data for the WRL.

TABLE 3
PROBABILISTIC GROUND MOTION DATA

PROBABILISTIC ESTIMATE EXPOSURE PERIOD (YEARS)	PROBABILITY OF EXCEEDANCE (%)	RETURN PERIOD (YEARS)	ESTIMATED PEAK HORIZONTAL GROUND ACCELERATION (G)
50	10	477	0.211
50	2	228	0.435

4.5 Design Basis Earthquake Event

Historically, the site experienced an estimated acceleration of 0.10 g during the event of March 12, 1934, which was the most critical for the site. Based on the risks associated with the Stansbury Fault, a site acceleration of 0.436 g is considered possible. From the probabilistic evaluation, a peak horizontal ground acceleration of 0.435 g was estimated for a 2% probability of exceedance in a 50 year exposure period.

Seed (1979) suggested that to ensure that displacements will be acceptably small, it is only necessary to perform a pseudo-static screening analysis for a seismic coefficient of 0.1 g for earthquakes up to a magnitude 6.5 or 0.15 g for earthquakes up to a magnitude 8.5, and obtain a factor of safety of 1.15 or greater. This procedure is only acceptable for site soils that are not vulnerable to excessive strength loss or pore pressure development. Both field and laboratory experience indicate that clayey soils, dry sands and in some cases dense saturated sands will not lose substantial resistance to deformation as a result of earthquake loading (Seed, 1979).

As described previously, the WRL subsurface consists mainly of clays, silts and fine sand at the lower elevation portions of the site, with more granular material at the higher elevation portions. Based on the Geotechnical Investigation Permit Modification prepared by AGEC (2004), water was encountered in the deeper borings at an approximate elevation of 4,220 ft to 4,335 ft (approximately 100 ft below the surface). These site subsurface conditions indicate that significant pore pressure generation is not a concern, and that Seed's (1979) procedure can be applied as an acceptable method of ensuring adequate performance for the WRL.

Based on the seismic hazard analyses and on Seed's (1979) procedure, the design earthquake we have chosen for this site would be from a magnitude 6.9 event on the

Stansbury fault. Therefore, a site horizontal seismic coefficient, k_h, of 0.15g was chosen based on Seed (1979) to be used as a pseudo-static screening value.

5.0 STABILITY ANALYSIS

5.1 General

Vector conducted stability analyses for the WRL for both static and pseudo-static conditions. Pseudo-static analyses were performed to determine the pseudo-static screening factor of safety and the yield acceleration for the slope condition analyzed. Failure surfaces through the waste and along the geomembrane liner were evaluated to determine the factor of safety for slope stability. Cross-section A-A' is located in the northern portion of the WRL, as shown on Figure 3 in the drawings. This section was chosen to present the most critical slope for the slope stability analyses. The analyzed cross section is presented in Appendix D.

The computer program SLIDE 5, developed by Rocscience, Inc (2003), was used for the analyses to determine the factors of safety and probabilities of failure. Spencer's Method of slices was used in the analysis to obtain the factor of safety. The factor of safety can be defined generally as the resisting forces divided by the driving forces. A factor of safety of 1.0 or less indicates that the slope is potentially unstable. Several search routines were used to evaluate tens of thousands of potential failure surfaces for each case analyzed.

Both static and pseudo-static analyses were performed for circular and non-circular surfaces. The pseudo-static analyses subject the two-dimensional sliding mass to a horizontal acceleration equal to a horizontal earthquake coefficient, k_h, multiplied by the acceleration of gravity. As described in section 4.5, a k_h of 0.15 was used as a screening tool for the slope stability evaluation of the WRL.

An infinite slope analysis was conducted for the proposed 2.5-foot thick Evapotranspirative (ET) cover system, to be constructed with "soil #2" material (see Vector Engineering report "Evaportranspirative (ET) Final Cover Permitting Report," 2006) for the 4H:1V side slopes. The Infinite Slope Method is commonly used for landfill cover analyses, and can incorporate the effects of landfill gas pressure, water buildup, and seismic events. A friction angle of 30 degrees was assumed for the cover soil based on laboratory strength test data (AGEC, 2004) with no adhesion. No landfill gas pressure was assumed because of the nature of the ET cover system. The infinite slope stability analyses method can account for the affects of cover soil saturation, as this can often cause cover systems to fail. The ET cover system proposed for this site is designed to remain partially saturated and is not intended to become fully saturated. A peak horizontal ground acceleration of 0.15 g was used for the Seed (1979) screening procedure, to determine if displacement analyses were required, as detailed in section 4.5 of this report.

5.2 Material Properties

The material properties of the various components of the landfill needed to perform static and pseudo-static slope stability analyses (e.g. unit weight and shear strength parameters) were obtained from the literature (Mitchell et al. 1992) and the previously performed interface shear testing. Table 4 shows a summary of the material properties used for the analyses.

TABLE 4
SUMMARY OF MATERIAL PROPERTIES USED IN STABILITY ANALYSES

SLOPE LINER SYSTEM	ANALYZED CRITICAL INTERFACE	TOTAL UNIT WEIGHT (PCF)	COHESION (PSF)	INTERNAL ANGLE OF FRICTION (DEGREES)
	Compacted Fill (Subgrade)	120	40	31
	Municipal Solid Waste (MSW)	65	100	30
Side Slope Liner GCL vs. Double Textured HDPE Geomembrane	Textured HDPE Geomembrane/ GCL	100	226[A]	14[A]
Floor Liner - Option 1 GCL vs. Double Smooth HDPE Geomembrane vs. Single Sided Geocomposite	Smooth HDPE Geomembrane/ Single Sided Geocomposite	100	20[A]	12[A]

SLOPE LINER SYSTEM	ANALYZED CRITICAL INTERFACE	TOTAL UNIT WEIGHT (PCF)	COHESION (PSF)	INTERNAL ANGLE OF FRICTION (DEGREES)
Floor Liner – Option 2 GCL vs. Double Textured HDPE Geomembrane vs. Single Sided Geocomposite	Textured HDPE Geomembrane / Single Sided Geocomposite	100	60[A]	15[A]
ET Final Cover *4H:1V Side Slopes*	Compacted Fill (ET cover)	100	0	30

A – From statistical analysis based on typical laboratory test results from similar liner interfaces.

5.3 Probabilistic Analysis

Variations in the strength parameters (i.e. cohesion and friction angle) can compromise the stability of the slopes. Slope stability analyses using worst-case strength parameters results in an overly conservative design. However, using mean strength parameters may result in an artificially high FOS. The probabilistic approach defines a range and statistical distribution for the soil strength parameters and densities used in the slope stability analyses. For each slip surface analyzed, a distribution of calculated safety factors is determined and a probability of failure is calculated. This approach accounts for the variability of the soil properties within the slope as shown in the field and laboratory test data.

The computer program SLIDE 5 (Rocscience, 2008) uses statistical distributions (i.e. Normal, Log Normal, Exponential, etc.) to model the variation in material properties in order to develop a Probability of Failure (PF) for a slope. For the WRL slope stability analyses, limited information was known about the shear strength of the geosynthetic/soil interface. From past experiences with similar interfaces, we selected the "most likely" shear strength properties for the interface at WRL. These properties were selected as the mean values for normally distributed data sets. The normal probability distribution function insures that 68% of the random values Slide selects for the shear strength properties of the interface, should fall within one standard deviation and the mean, and 95% of the random values should fall within two standard deviations of the mean. Standards of deviation for each of the material properties were determined from a database of strength tests on similar interfaces. Table 5 below summarizes the probabilistic material properties used for our analyses.

TABLE 5
SUMMARY OF PROPERTIES USED FOR PROBABILISTIC STABILITY ANALYSIS

MATERIAL	PROPERTY	DISTRIBUTION	MEAN	STD. DEV.	MIN	MAX
Interface	Cohesion (psf)	Normal	60	211	0	410
Interface	Phi (deg)	Normal	15	7	9	23

5.3 Results of the Stability Analyses

Circular and non-circular surfaces along the waste and liner interface, respectively, were evaluated using Spencer's method as well as a probabilistic approach. For the probabilistic slope stability analysis, statistical distributions to the model material properties (input parameters), such as cohesion and angle of friction, were assigned. These parameter values were based on laboratory test results for similar interfaces from tests conducted by Vector at our laboratory in Grass Valley, CA. This allowed the analyses to account for a degree of uncertainty in the cohesion and friction angle values for the geosynthetic interfaces.

The results of the stability analyses are summarized in Table 5. The critical failure surfaces originated near the toe of the waste slopes and day-lighted near the crest. The output presents the material properties, and locations of the critical shear surfaces with the lowest factor of safety (see Appendix D). The minimum factor of safety calculated in the pseudo-static analyses for the two liner system options was 0.91. Based on these results, seismic displacement analyses were performed.

The yield acceleration (k_y) of the landfill mass was calculated for both liner system configurations. The yield acceleration is defined as the horizontal acceleration that, when applied to the slope in the limit equilibrium (seismic) analyses, results in a pseudo-static factor of safety equal to one. The yield acceleration was determined using the Spencer method and the results are shown in Table 5. The output files from SLIDE 5 for these analyses are included in Appendix D.

The static factors of safety for the infinite slope stability analyses were between 2.8 and 3.0, meeting the accepted 1.5 FOS standard for lined MSW landfills. The pseudo-static (earthquake) factors of safety were between 1.7 and 1.8, greater than the 1.15 screening FOS specified by the Spencer (1979) procedure. The cover stability analysis and results are included in Appendix D.

TABLE 6
SUMMARY OF SLOPE STABILITY RESULTS FOR CROSS *SECTION A-A'*

	CASE ANALYZED	STATIC FACTOR OF SAFETY	STATIC PROBABILITY OF FAILURE (%)	PSEUDO-STATIC FACTOR OF SAFETY (K_h=0.15)	YIELD ACCEL. (Ky)
Non Circular Analysis	*Option 1* Smooth HDPE Geomembrane/ Single Sided Geocomposite	1.70	< 1	0.91	0.123
	Option 2 Textured HDPE / Single Sided Geocomposite	1.99	< 1	1.09	0.175
Circular Analysis	*Option 1* Smooth HDPE Geomembrane/ Single Sided Geocomposite	2.773	<1	1.58	0.34
	Option 2 Textured HDPE / Single Sided Geocomposite	2.829	<1	1.61	0.35
Infinite Slope Analysis	*2.5' ET Cover System* 4H:1V side slopes	2.31	<1	1.39	0.29

NOTE: Both liner configuration options have the same side slope liner system (Textured HDPE Geomembrane vs. GCL) properties as well as the MSW and the subgrade properties.

5.4 Conclusions Regarding Slope Stability

A factor of safety equal to or greater than 1.50 and 1.15 is generally considered acceptable for static conditions and pseudo-static conditions, respectively. Under static conditions the section analyzed showed an acceptable factor of safety for all liner configuration options. However, during an earthquake, displacement is possible since the pseudo-static factor of safety was less than 1.15 in both liner configurations. Therefore, a displacement analysis was performed, as discussed in the next section, to determine the potential displacement of the waste mass. The seismic stability analyses of the final cover system resulted in a FOS greater than 1.15, indicating that significant deformations in the final cover are not expected during the design earthquake.

6.0 SEISMIC DISPLACEMENT ANALYSIS

6.1 General

Seismic displacement analyses were performed for cross-section A-A' to evaluate the permanent displacements which may occur during an earthquake. The method chosen for the analyses was the "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills" by Bray et al. (1998). This method uses chart solutions to estimate the displacement for earthquake accelerations which are greater than the yield acceleration. The design earthquake would have a magnitude of 6.9. Based on the earthquake hazard analyses, the design site acceleration would be from a near field event on the Stansbury Fault zone. This event would result in a peak horizontal ground acceleration (PHGA) of 0.436 g at the site. In theory, the landfill will displace during a seismic event when the site acceleration exceeds the yield acceleration. The yield acceleration for floor-liner Option 1 was 0.123 g. The yield acceleration for floor-liner Option 2 was 0.175 g. The analyses show that base sliding of the landfill during the design earthquake would result in top displacements for both options (1 and 2) would be less than 1. For lined landfills, displacements less than or equal to 12 inches are generally considered acceptable (Kavazanjian 1999).

7.0 CONCLUSIONS

Vector performed slope stability analyses for the WRL based on the conceptual design of the landfill, preliminary soils data and historical seismicity near the site. Circular and non-circular failure surfaces through the waste and the critical liner interface were evaluated to determine the factor of safety for stability. Infinite slope stability analyses were performed on the final cover system. For static conditions, the results of the stability analyses indicate that the landfill will remain stable for all liner system configurations and the final cover system. For the pseudo-static conditions, the factor of safety for slope stability drops below 1.15, and therefore, a displacement analysis was performed. The displacement estimated from the seismic analysis for the weaker liner condition (Option 1) ranged from 0.0 in. to 0.3 in., which is considered acceptable (Kavazanjian 1999). Displacements for Option 2 ranged from 0.0 in to 0.1 in. Pseudo-static analyses for the final cover system resulted in a FOS greater than 1.15 and significant deformations in the covers system are not expected.

8.0 LIMITATIONS

The recommendations presented in this report are based upon understanding of the project, a field investigation, and the information provided by WRL. This report was prepared in accordance with generally accepted soils and foundation engineering practices applicable at the time the report was prepared. Vector makes no other warranties, either expressed or implied, as to the professional opinions and conclusions provided.

9.0 REFERENCES

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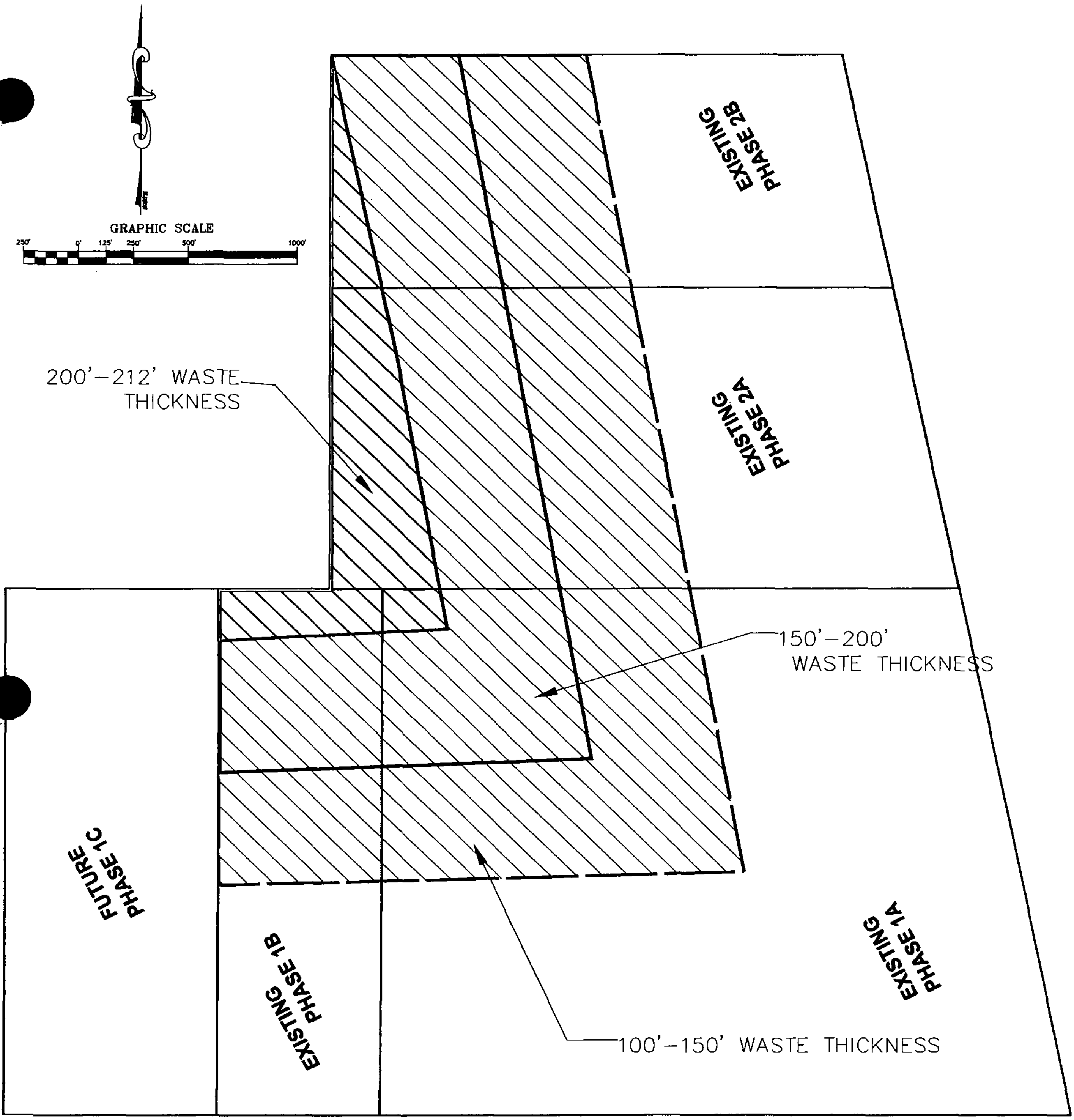


ISSUE: 12/03/2008 DESIGNED BY: DMW DRAWN BY: DMW CHECKED BY: JVR APPROVED BY: JVR	VECTOR ENGINEERING, INC. An Ausenco group company THE AMERICAS • ASIA • AUSTRALIA 143E Spring Hill Drive, Grass Valley, CA 95945 +1-530-272-2448 +1-530-272-8533 fax	PERMIT MODIFICATION WASATCH REGIONAL LANDFILL ALLIED WASTE, INC. TOOELE, UTAH **WASTE THICKNESS**	DRAWING NO. 01 PROJECT NO. 041200.00

APPENDIX A
LABORATORY TESTING

Vector Engineering Inc.
143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448
LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243A

Report Date: September 28, 2006
Client Name: ALLIED WASTE INC.
Project Name: WASATCH PHASE 1B
Project No: 061204.02

Superstrate:	⇐ Drainage Layer		
Material 1:	⇐ CETCO Bentomat GCL, Nonwoven side towards HDPE	LSN: AJB	Grip Board
Material 2:	⇒ GSE 60 mil HDPE Double textured, Roll# 108117461	LSN: AIT	Clamped
Substrate:	⇒ Concrete Board		

PEAK STRENGTH

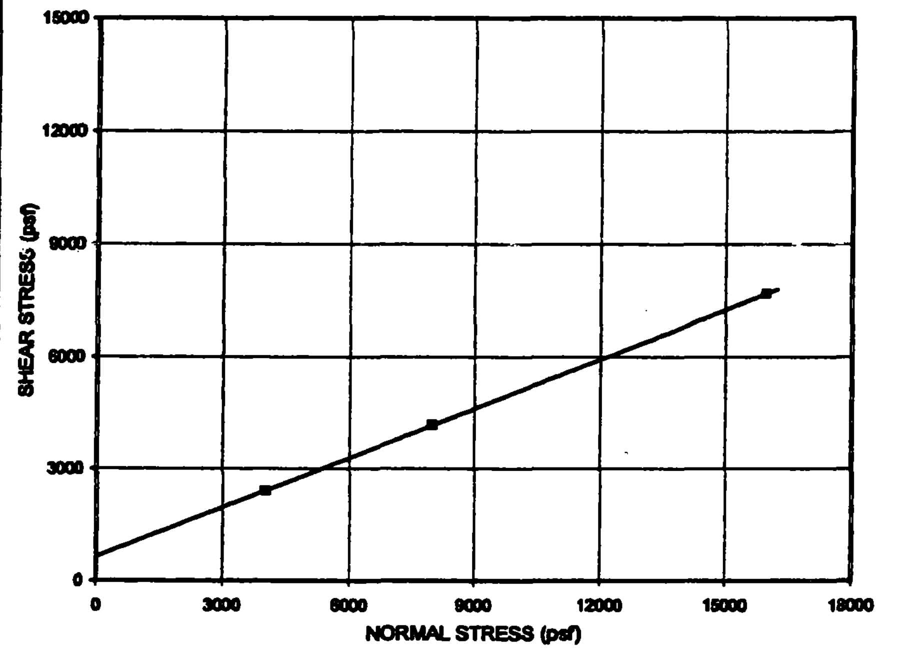
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	2400	31
2.	55.6	8000	4180	28
3.	111.1	16000	7680	26

Adhesion: 650 psf

Friction Angle: 24 degrees

Coefficient of Friction: 0.44

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 3.0 in. displacement)

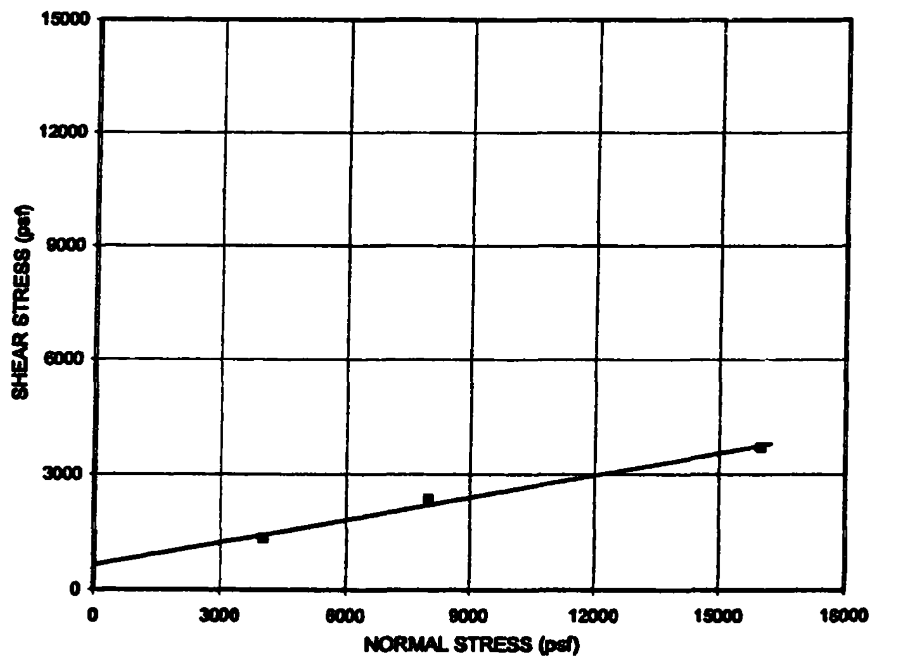
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	1320	18
2.	55.6	8000	2340	16
3.	111.1	16000	3700	13

Adhesion: 640 psf

Friction Angle: 11 degrees

Coefficient of Friction: 0.19

NOTE: GRAPH NOT TO SCALE



L:Labexcel \ Projects \ 2006 \ 061204 \ 1979A-LSDS-rp   Entered By: LM   Print Date: 10/13/06   Rev. By: Lab Log:

DCN: LSDS-rp (rev., 03/01/04)   **1979A**

Vector Engineering Inc.
143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448
LABORATORY SERVICES

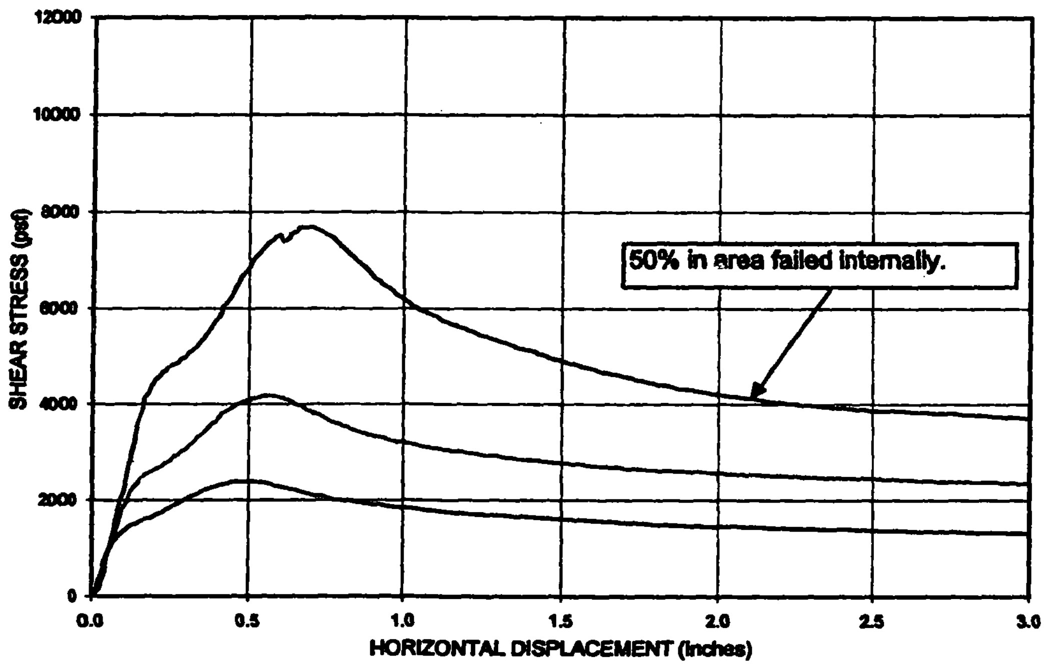
LARGE SCALE DIRECT SHEAR REPORT
Test Method D-6243A

Report Date: September 28, 2006
Client Name: ALLIED WASTE INC.
Project Name: WASATCH PHASE 1B
Project No: 061204.02

Superstrate: ⇦ Drainage Layer
Material 1: ⇦ CETCO Bentomat GCL, Nonwoven side towards HDPE — LSN: AJB — Grip Board
Material 2: ⇨ GSE 60 mil HDPE Double textured, Roll# 108117461 — LSN: AIT — Clamped
Substrate: ⇨ Concrete Board

DISPLACEMENT vs. SHEAR STRESS

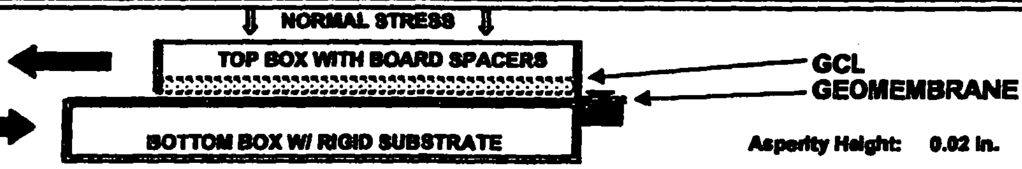
Test Point	Normal Stress psi	Normal Stress psf
1.	27.8	4000
2.	55.6	8000
3.	111.1	16000



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-6243 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



Asperity Height: 0.02 in.

SPECIAL TEST NOTES:

1. Each specimen of geomembrane was cut to 14" x 20" and clamped to the lower shear box.
2. Each specimen of GCL was cut to 12" x 12", then placed on the geomembrane and gripped using a grip board.
3. Each test point was consolidated for 24 hours at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred at the interface of the GCL and geomembrane specimens.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

Vector Engineering Inc.

143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448

LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243A

Report Date: September 28, 2006

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 1B Project No: 061204.02

Superstrate: ⇦ Drainage Layer

Material 1: ⇦ GSE 60 mil HDPE Smooth, Roll# 108117338 LSN: AIP Clamped

Material 2: ⇨ Claymax LSN: AJC Clamped

Substrate: ⇨ Concrete Board

PEAK STRENGTH

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	990	14
2.	55.6	8000	2060	14
3.	111.1	16000	4110	14

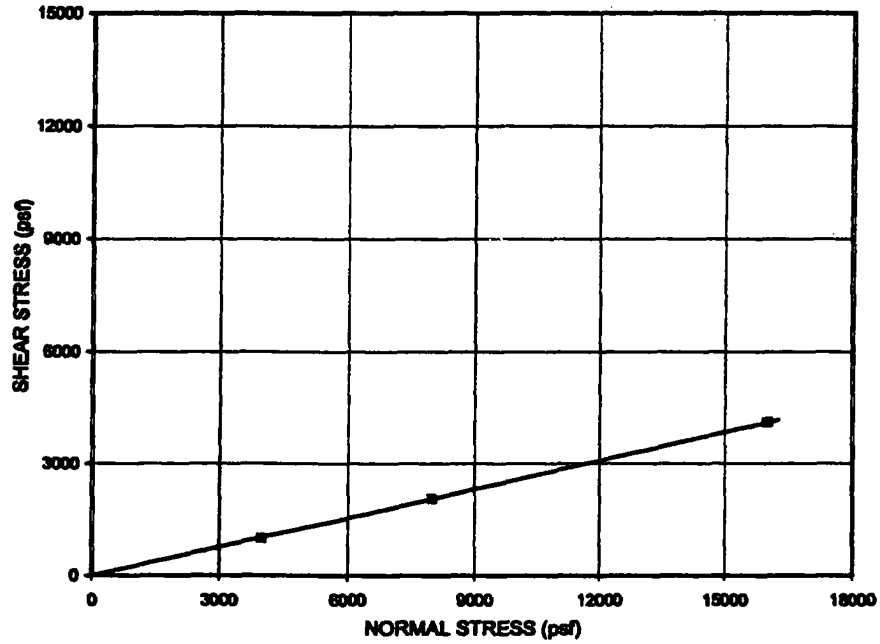
Adhesion: 0 psf

Friction Angle: 14 degrees

Coefficient of Friction: 0.26

Note: Intercept Adjusted to "0".

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 3.0 in. displacement)

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	610	9
2.	55.6	8000	1220	9
3.	111.1	16000	2530	9

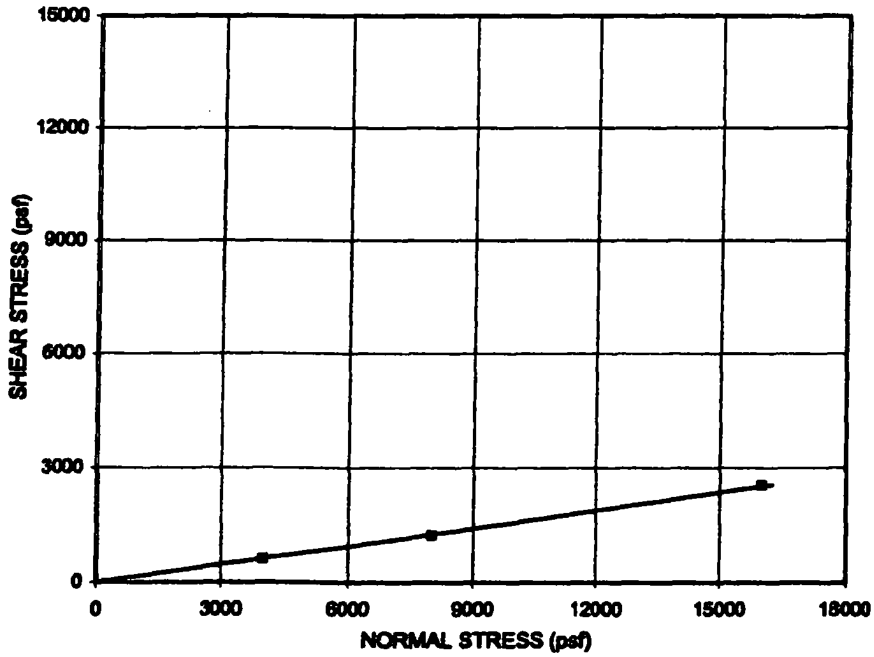
Adhesion: 0 psf

Friction Angle: 9 degrees

Coefficient of Friction: 0.16

Note: Intercept Adjusted to "0".

NOTE: GRAPH NOT TO SCALE



L:Labexcel \ Projects \ 2006 \ 061204 \ 1979B-LSDS-rp Entered By: LM Print Date: 10/13/06 Rev. By: Lab Log:

DCN: LSDS-rp (rev., 03/01/04) **1979B**

Vector Engineering Inc.

143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448

LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243A

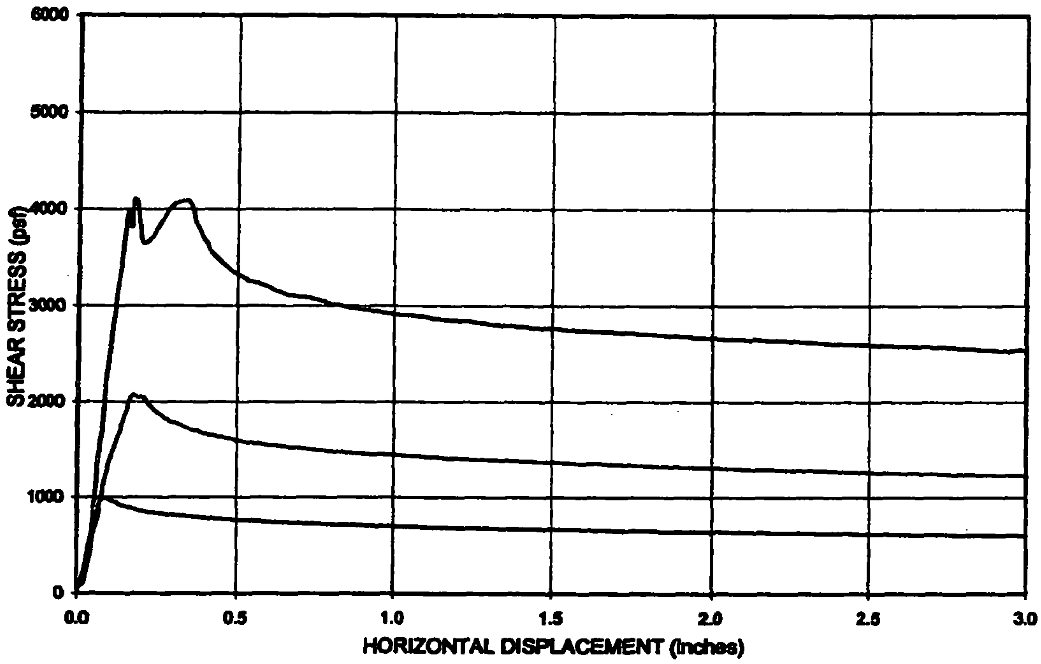
Report Date: September 28, 2006

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 1B Project No: 061204.02

Superstrate:	⟸ Drainage Layer			
Material 1:	⟸ GSE 60 mil HDPE Smooth, Roll# 108117338	LSN:	AIP	Clamped
Material 2:	⟹ Claymax	LSN:	AJC	Clamped
Substrate:	⟹ Concrete Board			

DISPLACEMENT vs. SHEAR STRESS

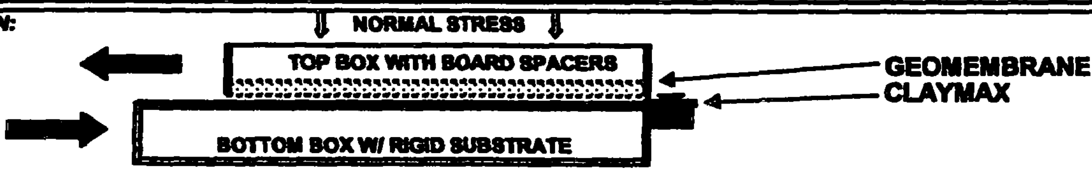
Test Point	Normal Stress psi	Normal Stress psf
1.	27.8	4000
2.	55.6	8000
3.	111.1	16000



STANDARD CONDITIONS: ***SHEAR DISPLACEMENT RATE: 0.04 in/min***

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-6243 using a Brainard-Kilman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. Each specimen of claymax was cut to 14" x 20" and clamped to the lower shear box.
2. Each specimen of geomembrane was cut to 12" x 12" and clamped to the upper shear box.
3. Each test point was consolidated for 24 hours at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred at the interface of the claymax and geomembrane specimens.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

Vector Engineering Inc.

143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448

LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-5321A

Report Date: September 25, 2006

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 1B Project No: 061204.02

Superstrata: ⇦	Drainage Layer		
Material 1: ⇦	GSE 60 mil HDPE Smooth, Roll# 108117338	LSN: AIP	Clamped
Material 2: ⇨	GSE Single textile Geocomposite, Roll# 131219846	LSN: AIS	Clamped
Substrata: ⇨	Concrete Board		

PEAK STRENGTH

Test Point	Normal Stress		Shear Stress	Secant Friction Angle
	psi	psf	psf	
1.	27.8	4000	1010	14
2.	55.6	8000	2150	15
3.	111.1	16000	4360	15

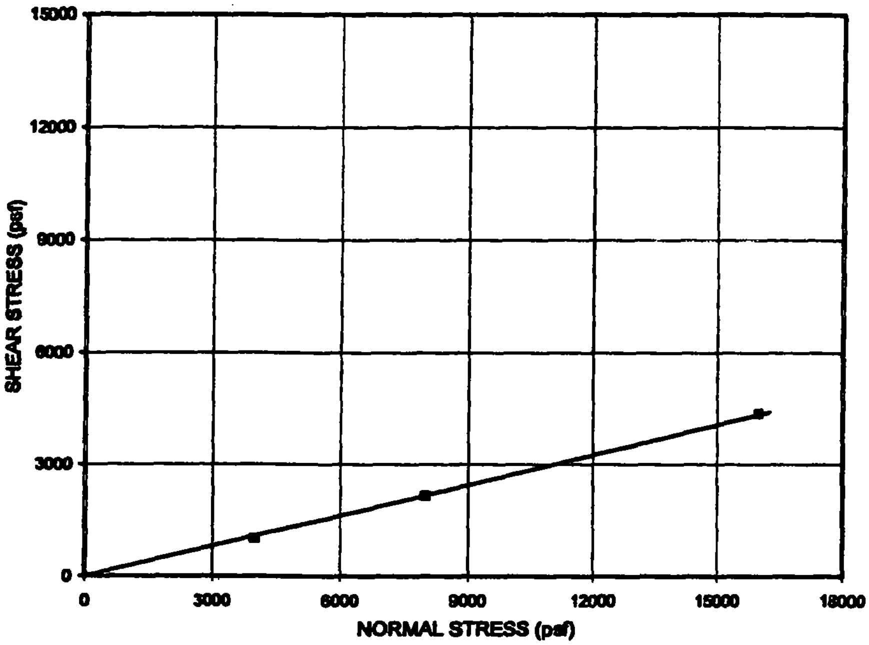
Adhesion: 0 psf

Friction Angle: 15 degrees

Coefficient of Friction: 0.27

Note: Intercept Adjusted to "0".

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 3.0 in. displacement)

Test Point	Normal Stress		Shear Stress	Secant Friction Angle
	psi	psf	psf	
1.	27.8	4000	630	9
2.	55.6	8000	1300	9
3.	111.1	16000	2860	10

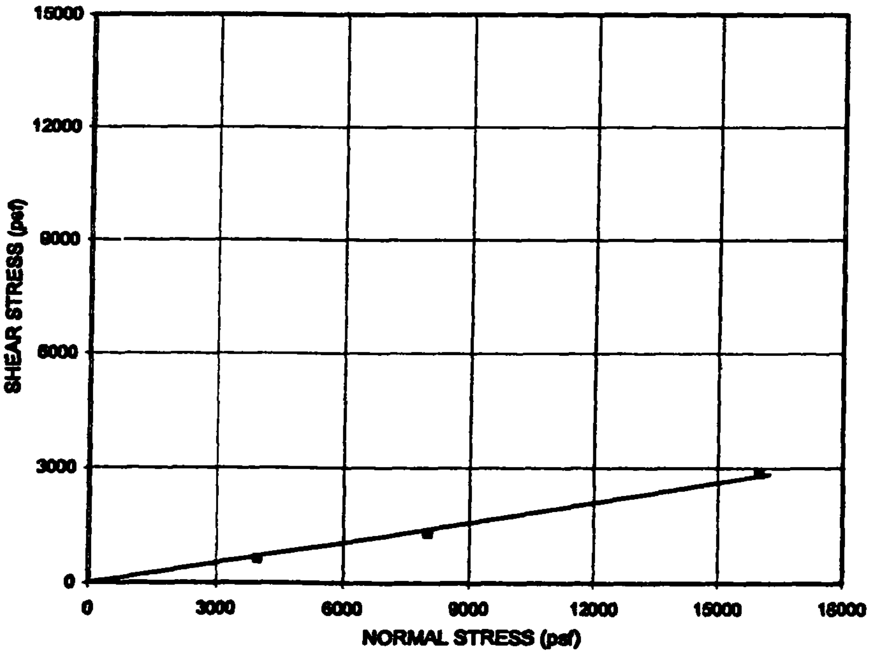
Adhesion: 0 psf

Friction Angle: 10 degrees

Coefficient of Friction: 0.17

Note: Intercept Adjusted to "0".

NOTE: GRAPH NOT TO SCALE



L:Labexcel \ Projects \ 2006 \ 061204 \ 1979C-LSDS-rp Entered By: LM Print Date: 10/13/06 Rev. By: Lab Log:

DCN: LSDS-rp (rev., 03/01/04) **1979C**

Vector Engineering Inc.

143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448

LABORATORY SERVICES

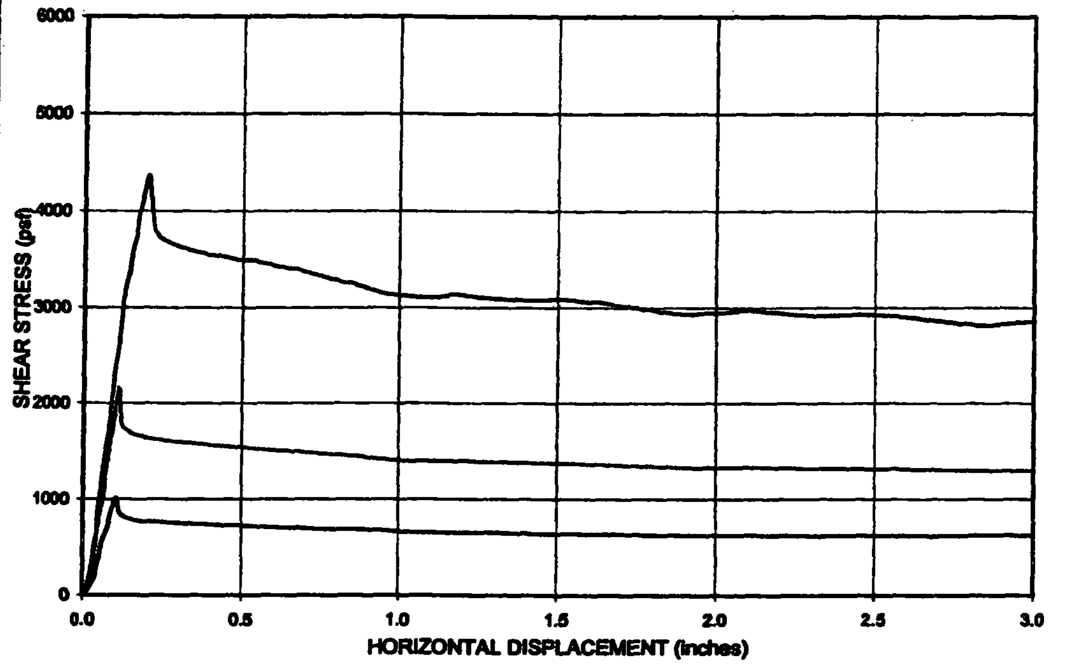
LARGE SCALE DIRECT SHEAR REPORT

Test Method D-5321A

Client Name:	ALLIED WASTE INC.	Project Name:	WASATCH PHASE 1B	Report Date:	September 25, 2006
				Project No:	061204.02
Superstrate:	⇐ Drainage Layer				
Material 1:	⇐ GSE 60 mil HDPE Smooth, Roll# 108117338		LSN: AIP	Clamped	
Material 2:	⇒ GSE Single textile Geocomposite, Roll# 131219846		LSN: AIS	Clamped	
Substrate:	⇒ Concrete Board				

DISPLACEMENT vs. SHEAR STRESS

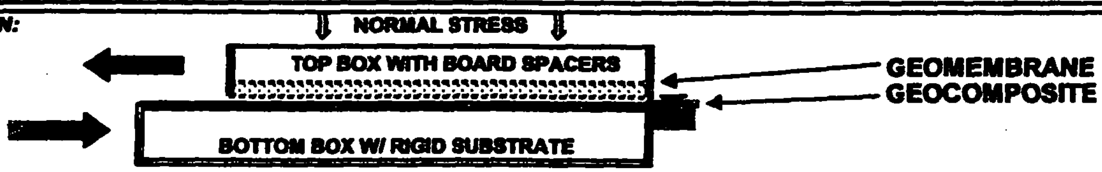
Test Point	Normal Stress psi	Normal Stress psf
1.	27.8	4000
2.	55.6	8000
3.	111.1	16000



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-5321 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. Each specimen of geocomposite was cut to 14" x 20" and clamped to the lower shear box.
2. Each specimen of geomembrane was cut to 12" x 12" and clamped to the upper shear box.
3. Each test specimen was consolidated for 1 hour at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred at the interface of the geocomposite and geomembrane specimens.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

Vector Engineering Inc.

143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448

LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243-A

Report Date: October 6, 2006

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 1B Project No: 061204.02

Superstrate:	⇦ Rigid Board			
Material 1:	⇦ Claymax	LSN:	AJC	Grip Board
Material 2:	⇨ Claymax	LSN:	AJC	Grip Board
Substrate:	⇨ Rigid Board			

PEAK STRENGTH

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	1470	20
2.	55.6	8000	2510	17
3.	111.1	16000	4500	16

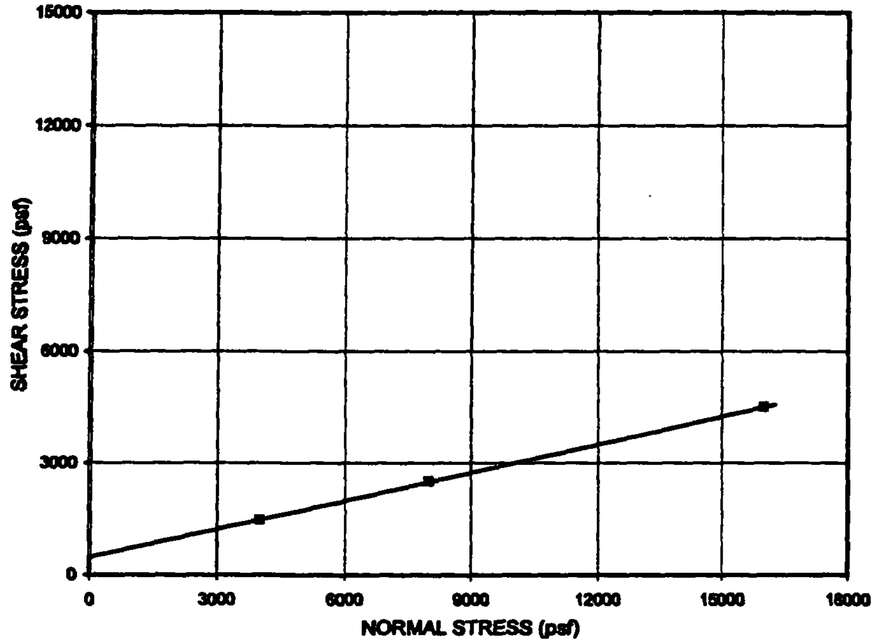
Adhesion: 470 psf

Friction Angle: 14 degrees

Coefficient of Friction: 0.25

Note: Intercept Adjusted to "0".

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 3.0 in. displacement)

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	650	9
2.	55.6	8000	1010	7
3.	111.1	16000	1630	6

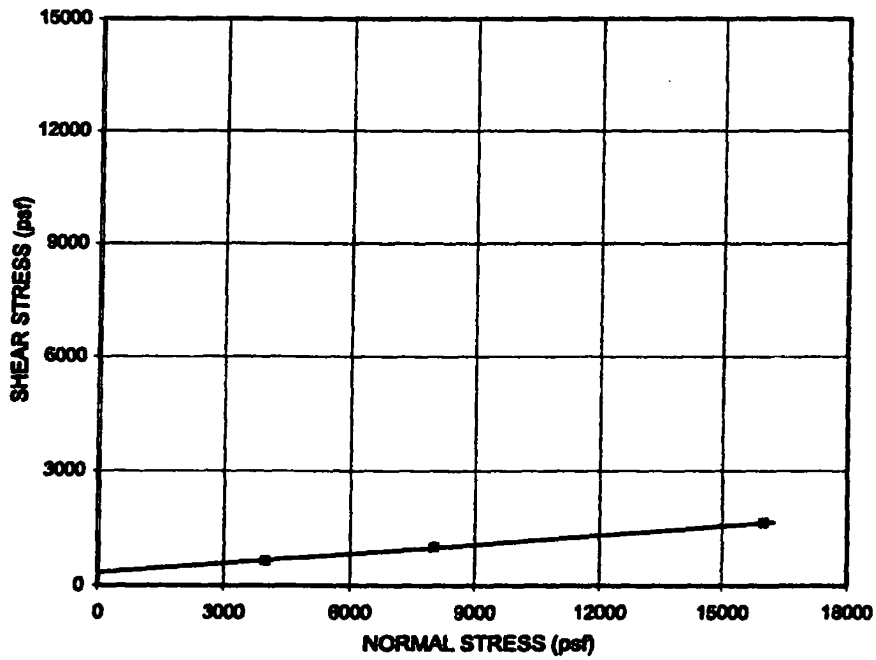
Adhesion: 340 psf

Friction Angle: 5 degrees

Coefficient of Friction: 0.08

Note: Intercept Adjusted to "0".

NOTE: GRAPH NOT TO SCALE



Vector Engineering Inc.

143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448

LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243-B

Report Date: October 6, 2006

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 1B Project No: 061204.02

Superstrata:	⇦ Rigid Board			
Material 1:	⇦ Claymax	LSN:	AJC	Grip Board
Material 2:	⇨ Claymax	LSN:	AJC	Grip Board
Substrata:	⇨ Rigid Board			

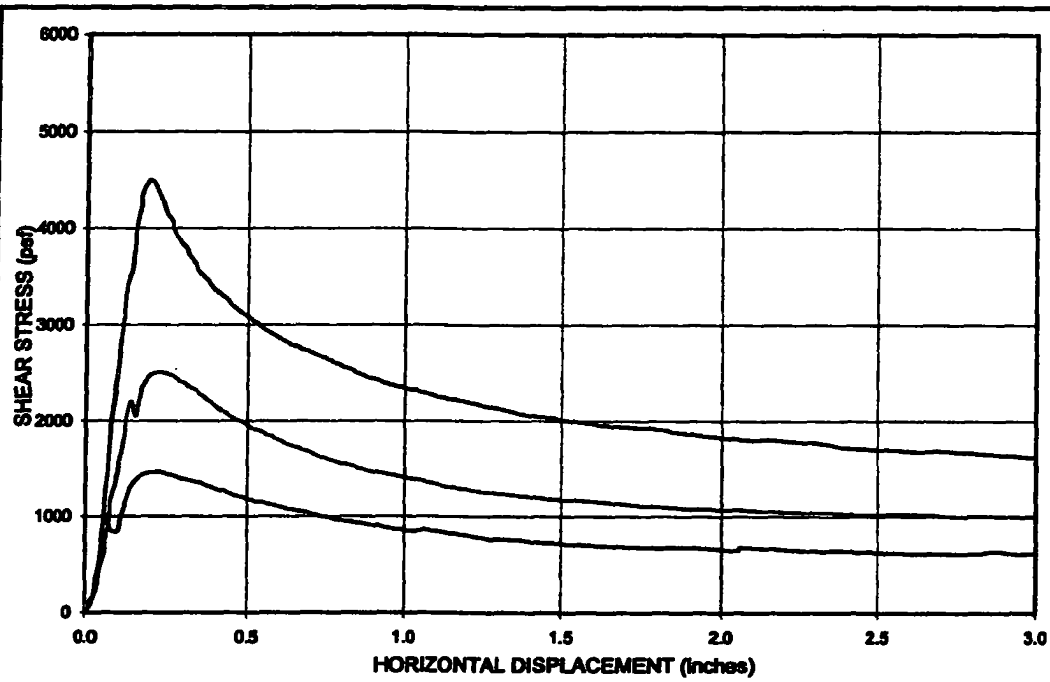
DISPLACEMENT vs. SHEAR STRESS

Test Point	Normal Stress psi	Normal Stress psf
1.	27.8	4000
2.	55.6	8000
3.	111.1	16000

MOISTURE DATA:

(GCL)

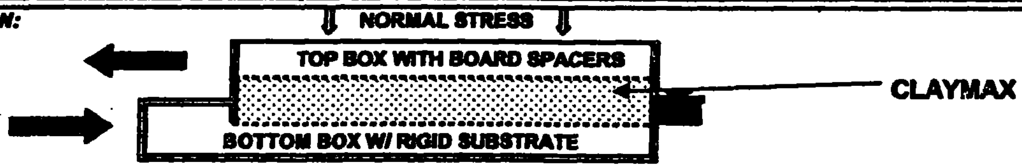
Final Water Content:(%)
1) 83.1 2) 69 3) 52.7



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-6243 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x 300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. Each specimen of claymax was cut to 12" x 12" and gripped using grip boards.
2. Each test point was consolidated for 24 hours at the specified normal stress, then sheared.
3. The test was performed in a "wet" or "flooded" condition.
4. Shearing occurred internally.
5. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
6. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

Vector Engineering Inc.

143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448

LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Special Shear - geosynthetic/geosynthetic

Report Date: April 29, 2008

Client Name: ALLIED WASTE INC Project Name: WASATCH REGIONAL LANDFILL PHASE 2B Project No: 061204.09

Superstrate: ⇦ Grip Board

Material 1: ⇦ CETCO GCL Bentomat ST Lot#2008 14LO Roll#1235 LSN: AOV Gripped

Material 2: ⇨ PolyFlex 60 mil HDPE T/T,Less Aggressive Side to GCL, Roll# HT1-6-07-148 LSN: AON Clamped

Substrate: ⇨ Concrete Board

PEAK STRENGTH

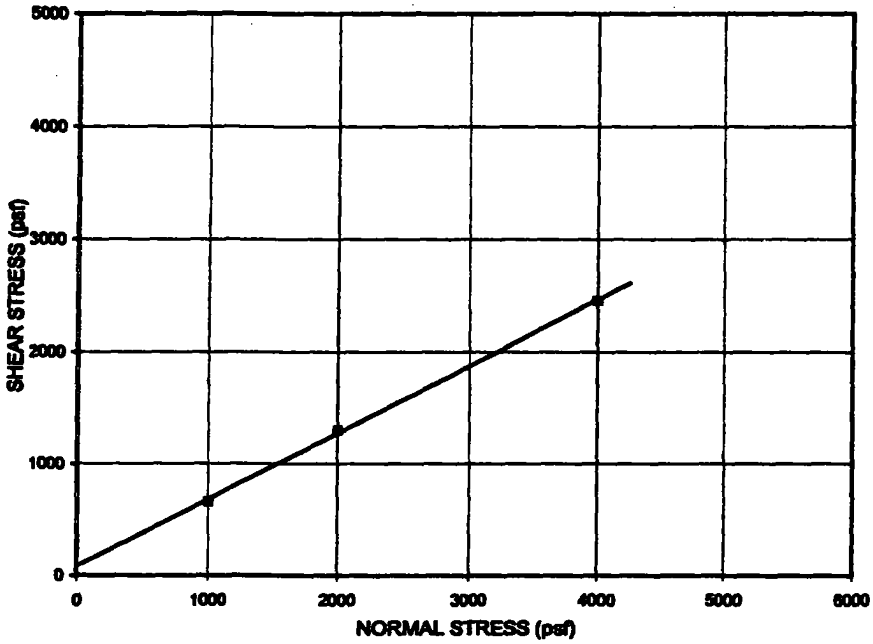
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	660	33
2.	13.9	2000	1300	33
3.	27.8	4000	2450	31

Adhesion: 80 psf

Friction Angle: 31 degrees

Coefficient of Friction: 0.6

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 2.5 in. displacement)

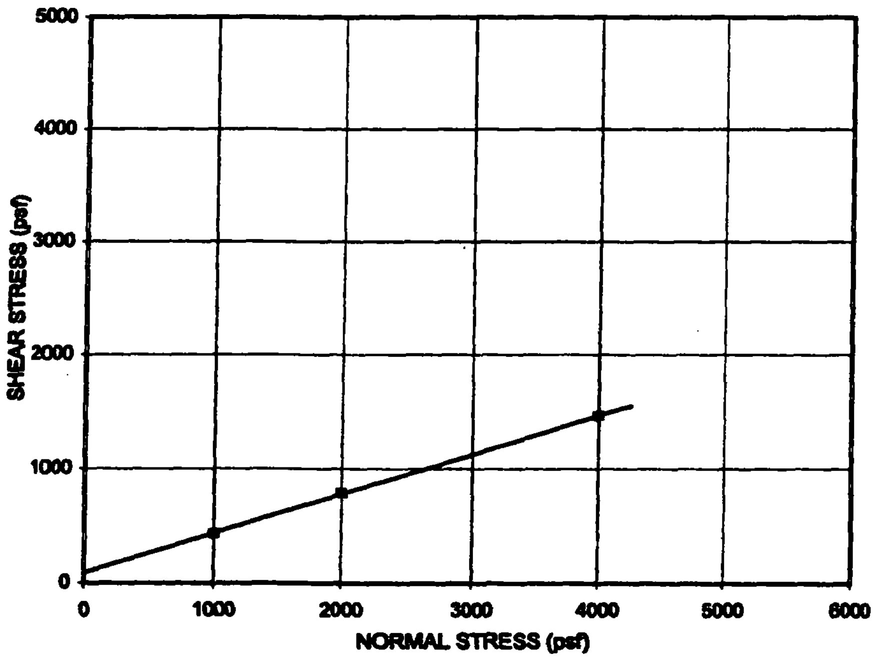
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	430	23
2.	13.9	2000	780	21
3.	27.8	4000	1460	20

Adhesion: 90 psf

Friction Angle: 19 degrees

Coefficient of Friction: 0.34

NOTE: GRAPH NOT TO SCALE



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Vector Engineering Inc.

143E Spring Hill Drive, Grass Valley, CA 95945 (530) 272-2448

LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Special Shear - geosynthetic/geosynthetic

Report Date: April 29, 2008

Client Name: ALLIED WASTE INC | Project Name: WASATCH REGIONAL LANDFILL PHASE 2B | Project No: 061204.09

Superstrate:	Grip Board			
Material 1:	CETCO GCL Bentomat ST Lot#2008 14LO Roll#1235	LSN:	AOV	Gripped
Material 2:	PolyFlex 60 mil HDPE T/T,Less Aggressive Side to GCL, Roll# HT1-6-07-148{	LSN:	AON	Clamped
Substrate:	Concrete Board			

DISPLACEMENT vs. SHEAR STRESS

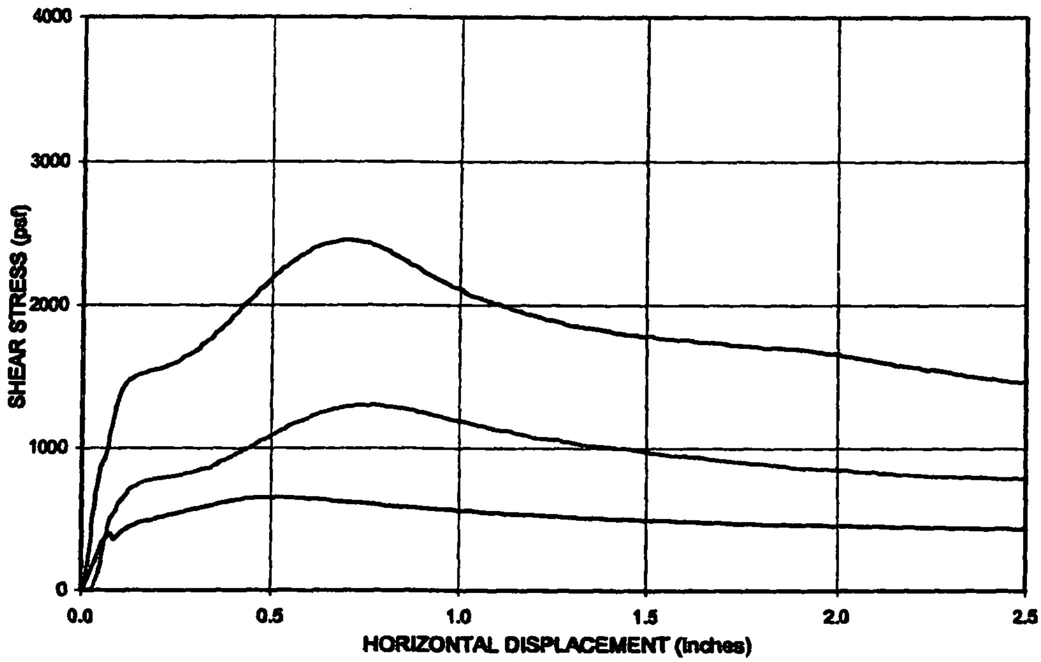
Test Point	Normal Stress psi	Normal Stress psf
1.	6.9	1000
2.	13.9	2000
3.	27.8	4000

MOISTURE DATA:

(GCL)

Initial Water Content 20%

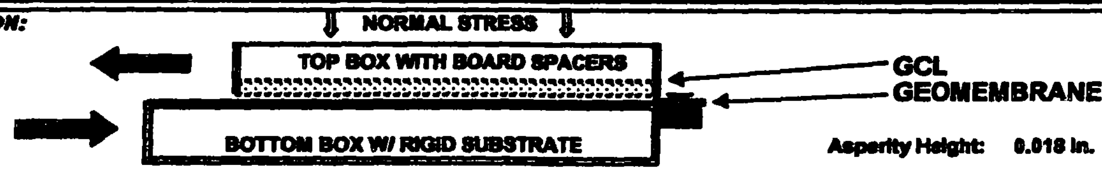
Final Water Content:(%)
1) 67.1 2) 60.5 3) 46.7



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-6243 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



Asperity Height: 0.018 in.

SPECIAL TEST NOTES:

1. Each specimen of geomembrane was cut to 14" x 20" and clamped to the lower shear box.
2. Each GCL specimen was cut to 12" x 12", gripped and placed into the upper shear box.
3. Each test specimen was consolidated for 24 hours at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred mainly at the interface of the GCL and geomembrane specimens.
6. Point 1 had .75 inches (white side bunched up) of internal shearing, point 3 sheared internally (2.5 inches white side bunched up).
7. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
8. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

Vector Engineering Inc.

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243-B

Report Date: April 29, 2008

Client Name: ALLIED WASTE INC | Project Name: WASATCH REGIONAL LANDFILL PHASE 2B | Project No: 061204.09

Superstrate:	⟸ Grip Board			
Material 1:	⟸ CETCO GCL Claymax 200R, Lot#2008 15LO, Roll#1640	LSN:	AOW	Gripped
Material 2:	⟹ PolyFlex 60 mil HDPE Smooth, Roll# HS2-6-08-0029-5	LSN:	AOL	Clamped
Substrate:	⟹ Concrete Board			

PEAK STRENGTH

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	930	13
2.	55.6	8000	1980	14
3.	111.1	16000	4110	14

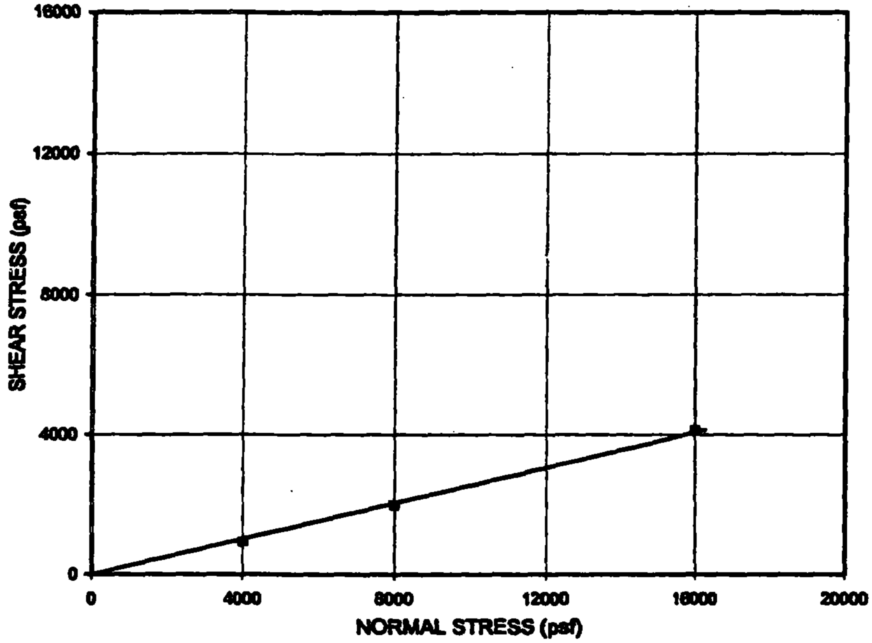
Adhesion: 0 psf

Friction Angle: 14 degrees

Coefficient of Friction: 0.25

Note: Intercept set to "0".

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 2.5 in. displacement)

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	610	9
2.	55.6	8000	1270	9
3.	111.1	16000	2580	9

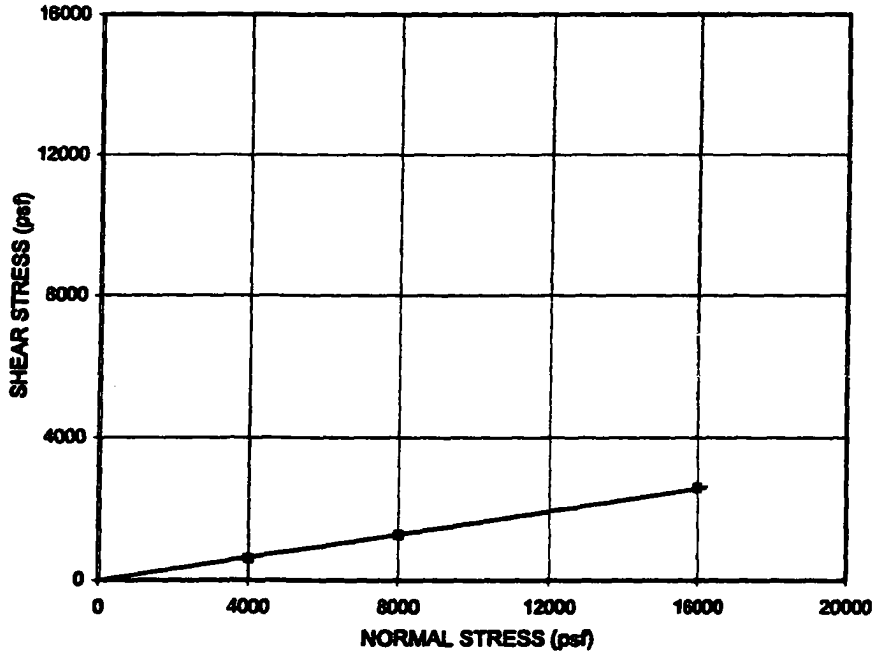
Adhesion: 0 psf

Friction Angle: 9 degrees

Coefficient of Friction: 0.16

Note: Intercept set to "0".

NOTE: GRAPH NOT TO SCALE



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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243-B

Report Date: April 29, 2008

Client Name: ALLIED WASTE INC — Project Name: WASATCH REGIONAL LANDFILL PHASE 2B — Project No: 061204.09

Superstrate: ⇐ Grip Board

Material 1: ⇐ CETCO GCL Claymax 200R, Lot#2008 15LO, Roll#1640 — LSN: AOW — Gripped

Material 2: ⇒ PolyFlex 60 mil HDPE Smooth, Roll# HS2-6-08-0029-5 — LSN: AOL — Clamped

Substrate: ⇒ Concrete Board

DISPLACEMENT vs. SHEAR STRESS

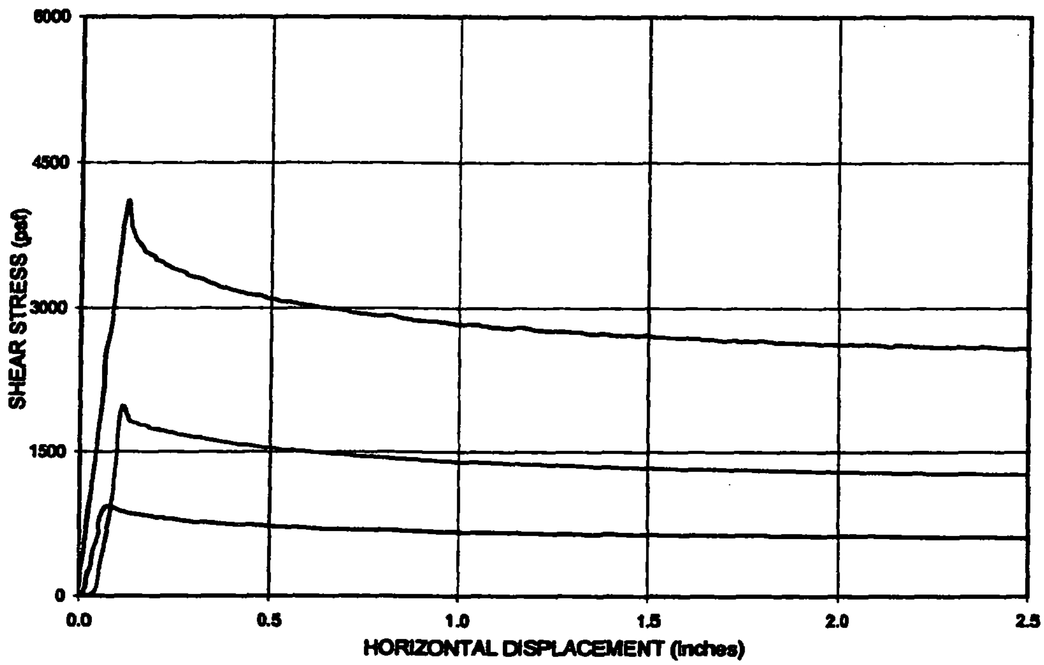
Test Point	Normal Stress psi	Normal Stress psf
1.	27.8	4000
2.	55.6	8000
3.	111.1	16000

MOISTURE DATA:

(GCL)

Initial Water Content: 44.77%

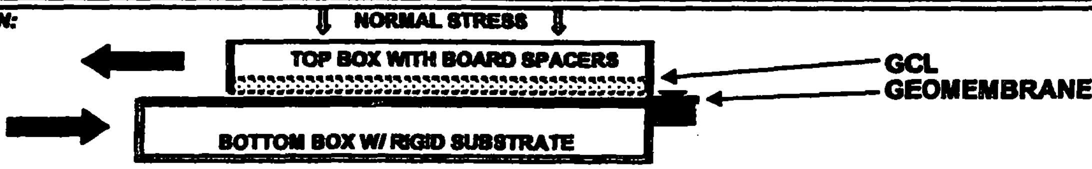
Final Water Content:(%) 1) 56.3 2) 47.6 3) 39.2



STANDARD CONDITIONS: — **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-6243 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. Each specimen of geomembrane was cut to 14" x 20" and clamped to the lower shear box.
2. Each GCL specimen was cut to 12" x 12", gripped and placed into the upper shear box.
3. Each test specimen was consolidated for 24 hours at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred mainly at the interface of the GCL and geomembrane specimens.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

Vector Engineering Inc.

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-5321A

Report Date: April 29, 2008

Client Name: ALLIED WASTE INC   Project Name: WASATCH REGIONAL LANDFILL PHASE 2B   Project No: 061204.09

Superstrate: ⇦ Board Spacers

Material 1: ⇦ PolyFlex 60 mil HDPE Smooth, Roll# HS2-6-08-0029-5   LSN: AOL   Clamped

Material 2: ⇨ SKAPS Single Sided Geocomposite, Roll# TN 220-1-8 (net to HDPE)   LSN: AOP   Clamped

Substrate: ⇨ Concrete Board

PEAK STRENGTH

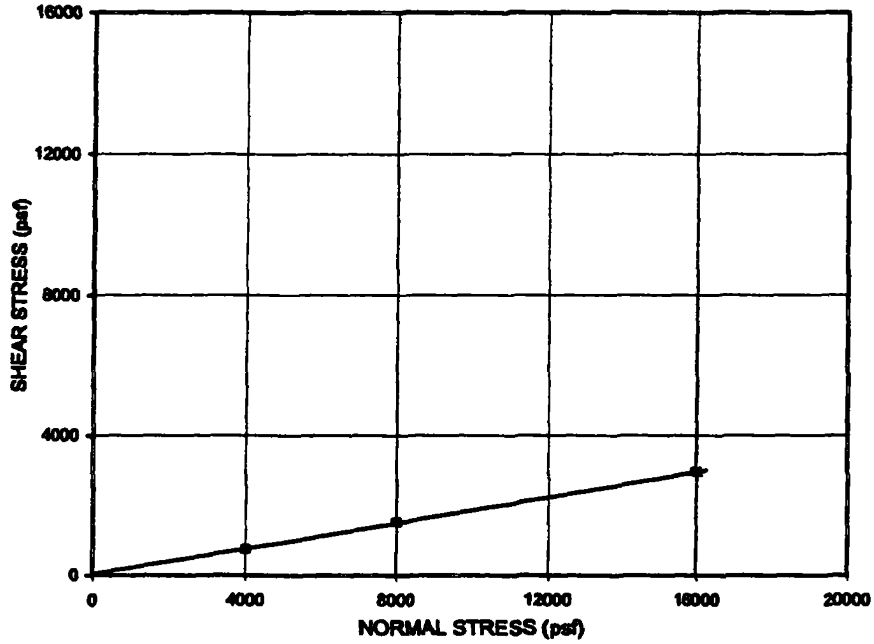
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	750	11
2.	55.6	8000	1520	11
3.	111.1	16000	2940	10

Adhesion: 40 psf

Friction Angle: 10 degrees

Coefficient of Friction: 0.18

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 2.5 in. displacement)

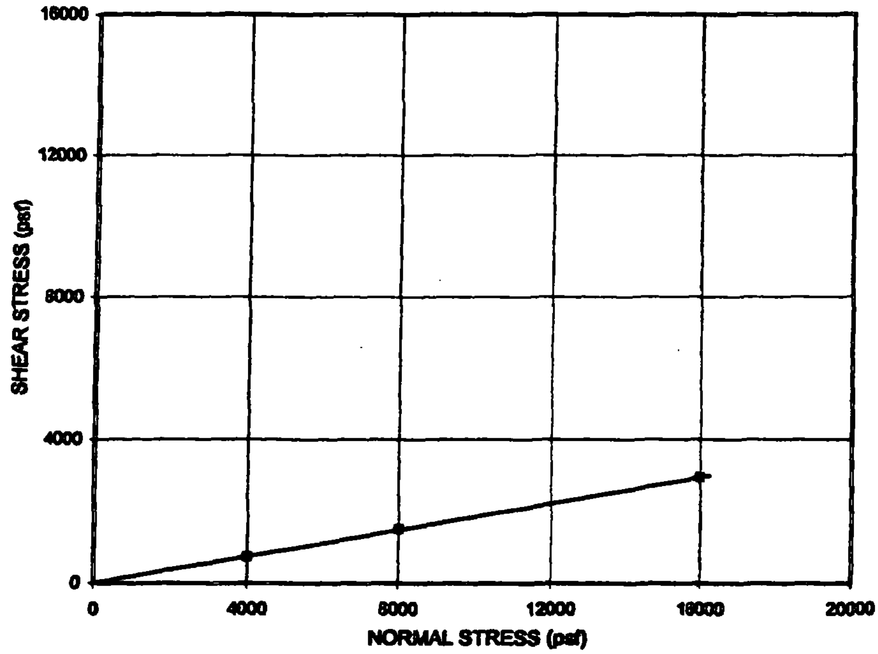
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	730	10
2.	55.6	8000	1510	11
3.	111.1	16000	2940	10

Adhesion: 20 psf

Friction Angle: 10 degrees

Coefficient of Friction: 0.18

NOTE: GRAPH NOT TO SCALE



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DCN: LSDS-rp (rev., 03/01/04)

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-5321A

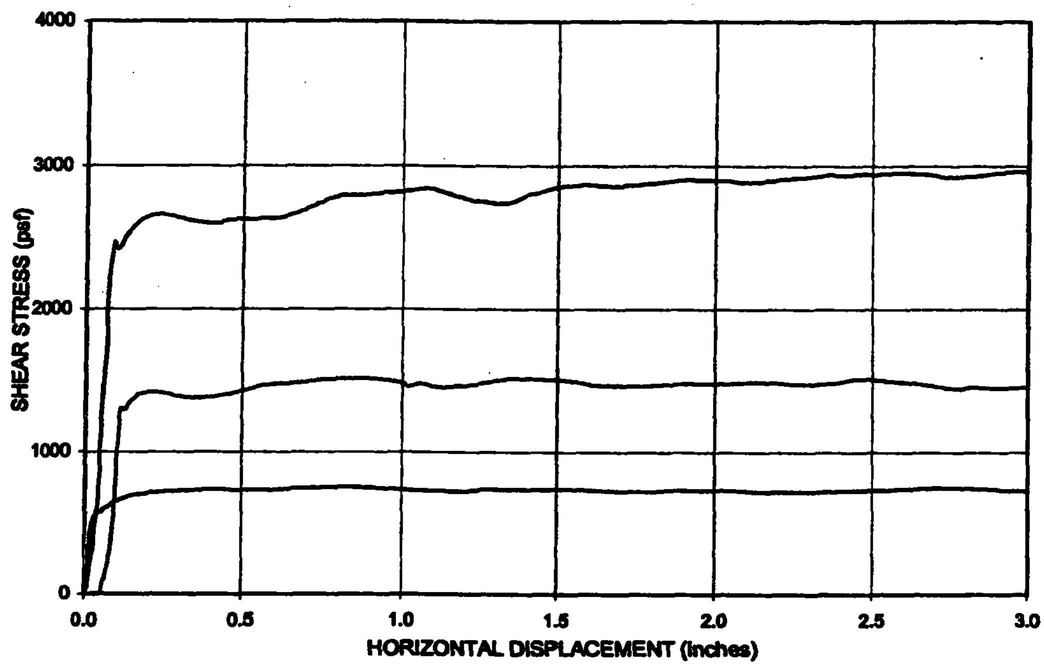
Report Date: April 29, 2008

Client Name: ALLIED WASTE INC | Project Name: WASATCH REGIONAL LANDFILL PHASE 2B | Project No: 061204.09

Superstrate:	⇐ Board Spacers		
Material 1:	⇐ PolyFlex 60 mil HDPE Smooth, Roll# HS2-8-08-0029-5	LSN: AOL	Clamped
Material 2:	⇒ SKAPS Single Sided Geocomposite, Roll# TN 220-1-8 (net to HDPE)	LSN: AOP	Clamped
Substrate:	⇒ Concrete Board		

DISPLACEMENT vs. SHEAR STRESS

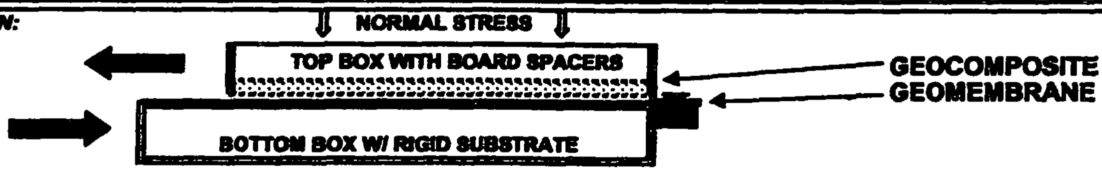
Test Point	Normal Stress psi	Normal Stress psf
1.	27.8	4000
2.	55.6	8000
3.	111.1	16000



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-5321 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. Each specimen of geomembrane was cut to 14" x 20" and clamped to the lower shear box.
2. Each specimen of geocomposite was cut to 14" x 16" and clamped to the upper shear box.
3. Each test specimen was consolidated for 1 hours at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred at the interface of the geomembrane and geocomposite specimens.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243-B

Report Date: April 9, 2007
Project No: 061204.05

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 2A

Superstrate: ⇦ Grip Board & Drainage Layer
Material 1: ⇦ GSE GCL Bentofix NS, Roll# 39932, Nonwoven side towards HDPE LSN: AKS Grip Board
Material 2: ⇨ GSE 60 mil HDPE Double textured, Roll# 103138468 LSN: ALH Clamped
Substrate: ⇨ Concrete Board

PEAK STRENGTH

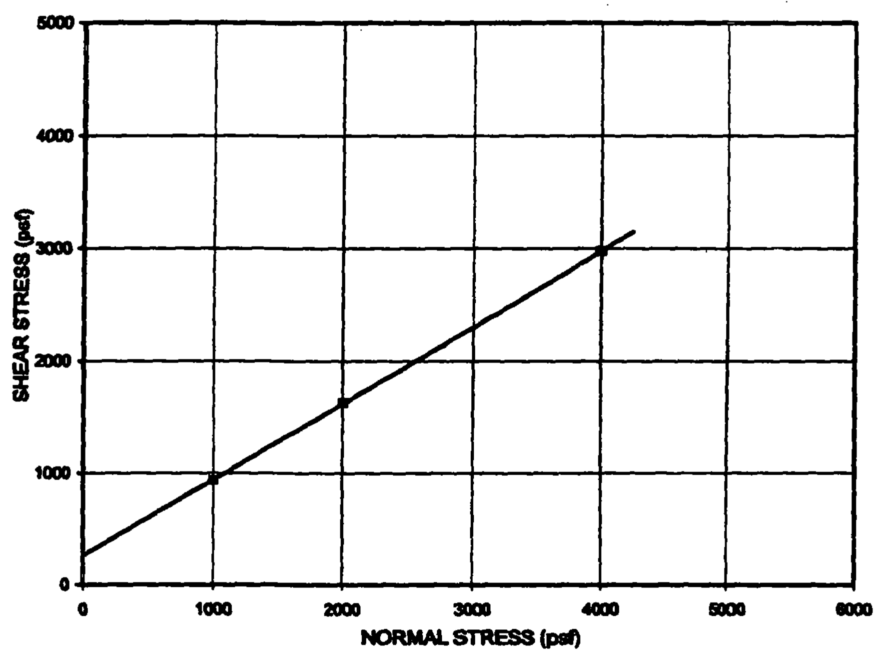
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	930	43
2.	13.9	2000	1620	39
3.	27.8	4000	2970	37

Adhesion: 260 psf

Friction Angle: 34 degrees

Coefficient of Friction: 0.68

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 2.5 in. displacement)

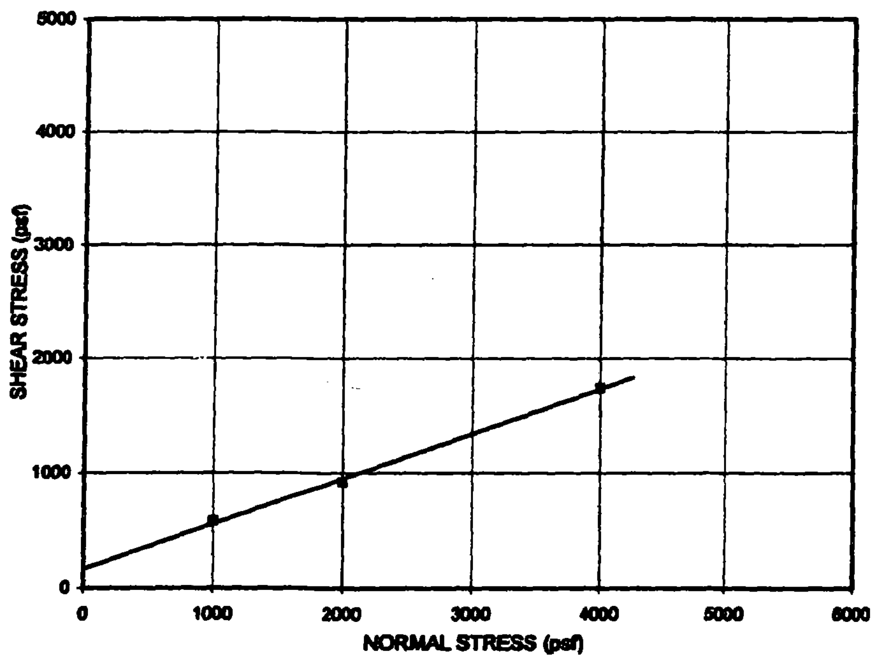
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	6.9	1000	580	30
2.	13.9	2000	920	25
3.	27.8	4000	1740	24

Adhesion: 160 psf

Friction Angle: 21 degrees

Coefficient of Friction: 0.39

NOTE: GRAPH NOT TO SCALE



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DCN: LSDS-rp (rev., 03/01/04) **2133A**

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243-B

Report Date: April 9, 2007
Project No: 061204.05

Client Name: ALLIED WASTE INC.   Project Name: WASATCH PHASE 2A

Superstrate: ⟸ Grip Board & Drainage Layer

Material 1: ⟸ GSE GCL Bentofix NS, Roll# 39932, Nonwoven side towards HDPE   LSN: AKS   Grip Board

Material 2: ⟹ GSE 60 mil HDPE Double textured, Roll# 103138468   LSN: ALH   Clamped

Substrate: ⟹ Concrete Board

DISPLACEMENT vs. SHEAR STRESS

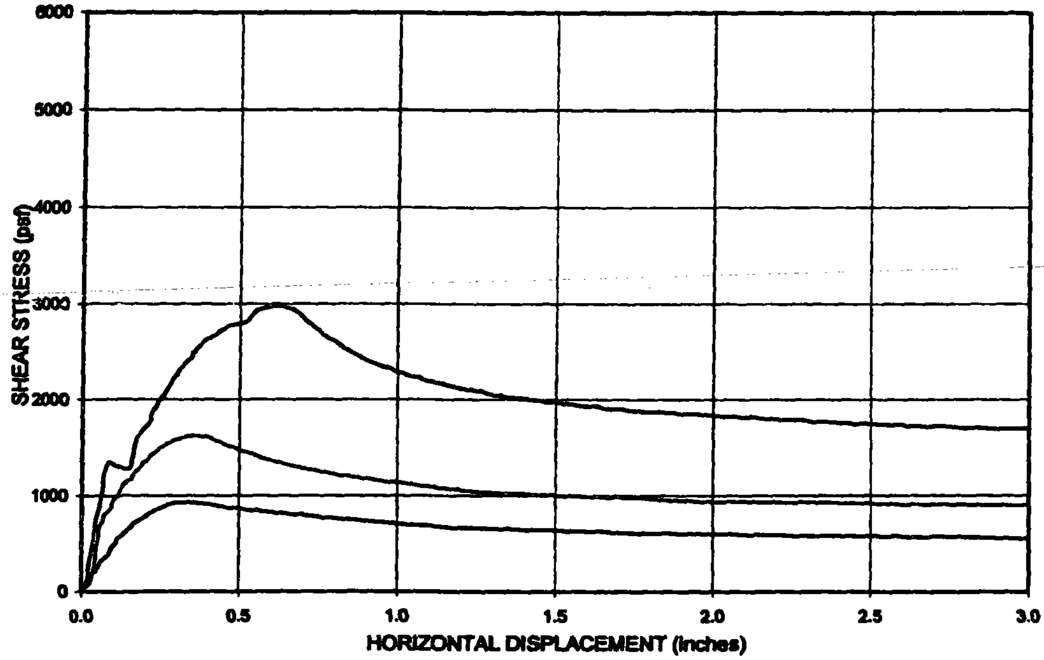
Test Point	Normal Stress psi	Normal Stress psf
1.	6.9	1000
2.	13.9	2000
3.	27.8	4000

MOISTURE DATA:

(GCL)

Initial Water Content: 7.5%

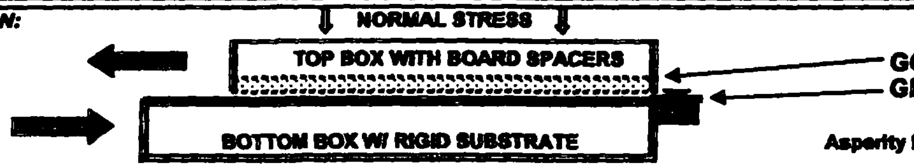
Final Water Content:(%)
1) 62.2 2) 60.4 3) 59.8



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-6243 using a Brainard-Killman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



Asperity Height: 0.022 in.

SPECIAL TEST NOTES:

1. Each specimen of geomembrane was cut to 14" x 20" and clamped to the lower shear box.
2. Each specimen of GCL was cut to 12" x 12", then placed on the geomembrane and gripped using a grip board.
3. Each test point was consolidated for 24 hours at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred at the interface of the GCL and geomembrane specimens.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243-B

Report Date: April 10, 2007

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 2A Project No: 061204.05

Superstrate: ⇦ Grip Board & Drainage Layer

Material 1: ⇦ GSE GCL Bentofix EC, Roll# 502100520, Nonwoven side towards HDPE LSN: ALI Grip Board

Material 2: ⇨ GSE 60 mil HDPE Smooth, Roll# 108120131 LSN: ALD Clamped

Substrate: ⇨ Concrete Board

PEAK STRENGTH

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	1180	16
2.	55.6	8000	2290	16
3.	111.1	16000	4890	17

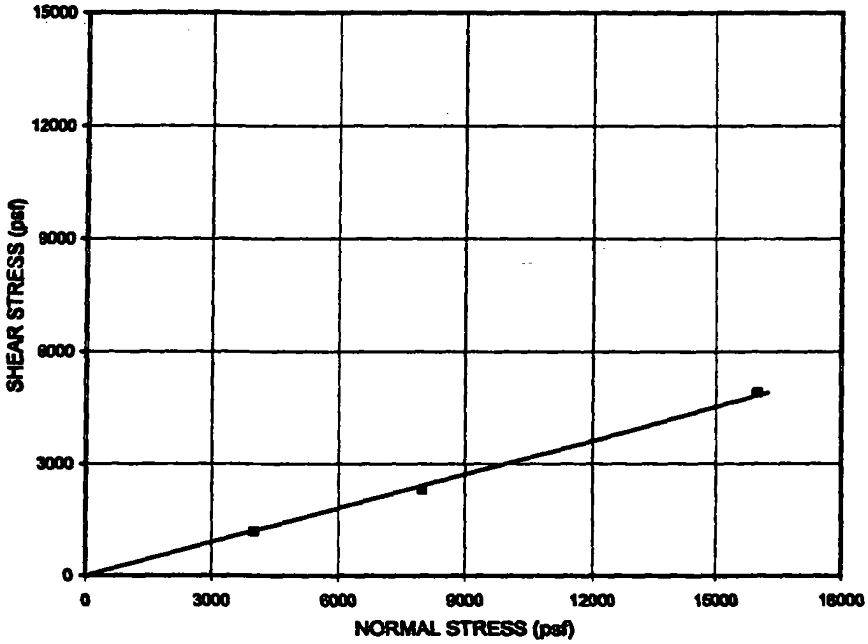
Adhesion: 0 psf

Friction Angle: 17 degrees

Coefficient of Friction: 0.3

Note: Intercept adjusted to 0.

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 2.5 in. displacement)

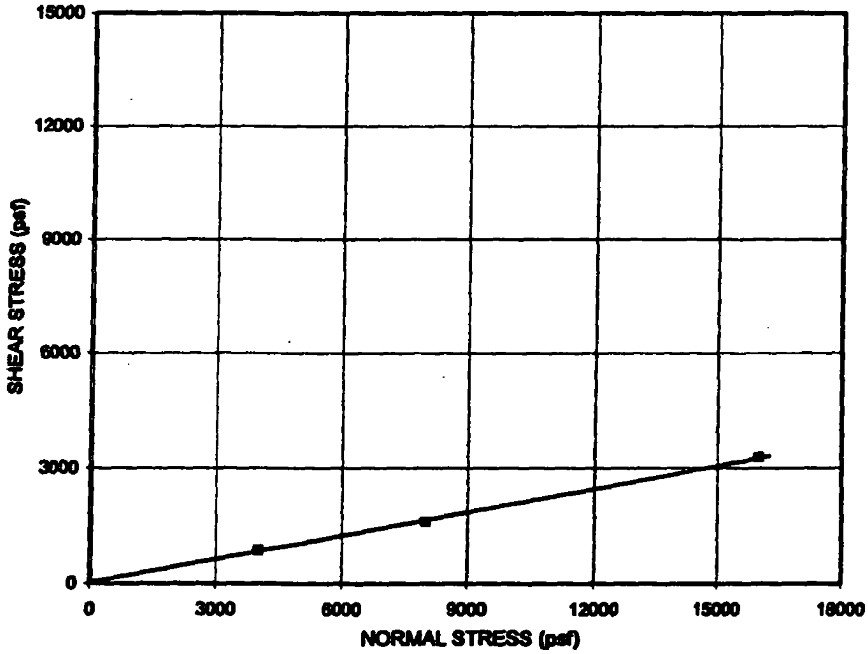
Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	870	12
2.	55.6	8000	1600	11
3.	111.1	16000	3280	12

Adhesion: 30 psf

Friction Angle: 11 degrees

Coefficient of Friction: 0.2

NOTE: GRAPH NOT TO SCALE



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DCN: LSDS-rp (rev., 03/01/04) **2133B**

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-6243-B

Report Date: April 10, 2007

Client Name: ALLIED WASTE INC. | Project Name: WASATCH PHASE 2A | Project No: 061204.05

Superstrate:	⇦ Grip Board & Drainage Layer			
Material 1:	⇦ GSE GCL Bentofix EC, Roll# 502100520, Nonwoven side towards HDPE	LSN:	ALI	Grip Board
Material 2:	⇨ GSE 60 mil HDPE Smooth, Roll# 108120131	LSN:	ALD	Clamped
Substrate:	⇨ Concrete Board			

DISPLACEMENT vs. SHEAR STRESS

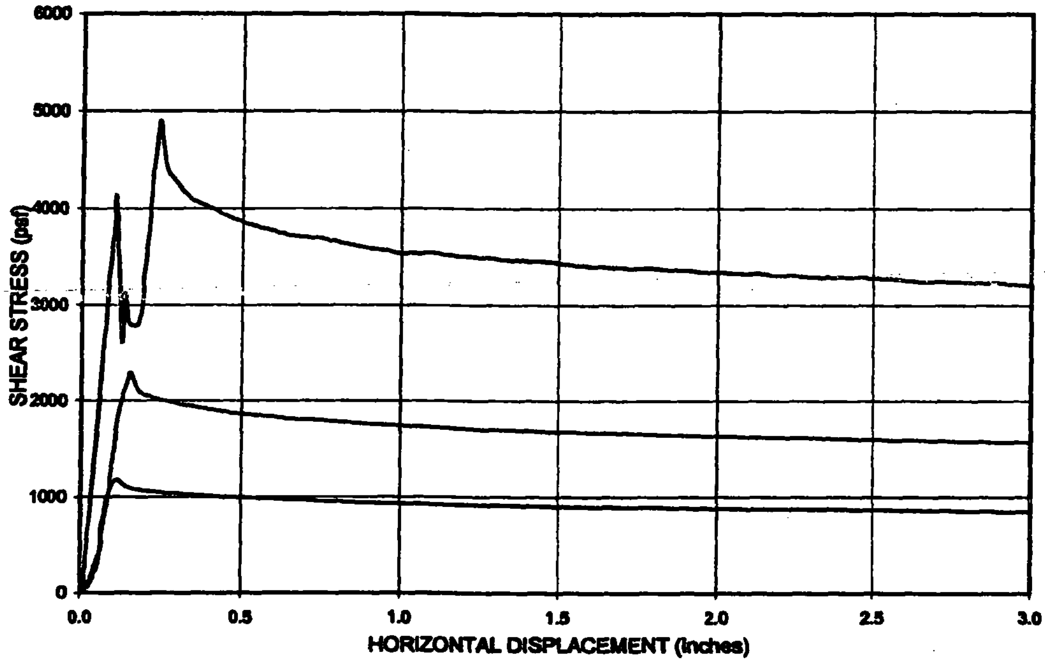
Test Point	Normal Stress psi	Normal Stress psf
1.	27.8	4000
2.	55.6	8000
3.	111.1	16000

MOISTURE DATA:

(GCL)

Initial Water Content: 9.8%

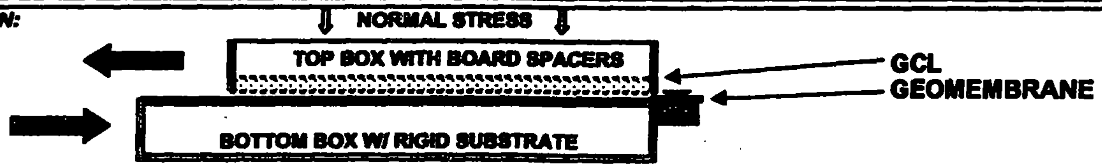
Final Water Content:(%)
1) 63.3 2) 60.6 3) 54.7



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-6243 using a Brainard-Kilman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. Each specimen of geomembrane was cut to 14" x 20" and clamped to the lower shear box.
2. Each specimen of GCL was cut to 12" x 12", then placed on the geomembrane and gripped using a grip board.
3. Each test point was consolidated for 24 hours at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred at the interface of the GCL and geomembrane specimens.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

Vector Engineering Inc.

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-5321A

Report Date: April 6, 2007

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 2A Project No: 061204.05

Superstrata: Drainage Layer

Material 1: GSE 60 mil HDPE Smooth, Roll# 108120131 LSN: ALD Clamped

Material 2: GSE Single side textile Geocomposite, Roll# 131238484 LSN: ALG Clamped

Substrata: Concrete Board

PEAK STRENGTH

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	950	13
2.	55.6	8000	1860	13
3.	111.1	16000	4380	15

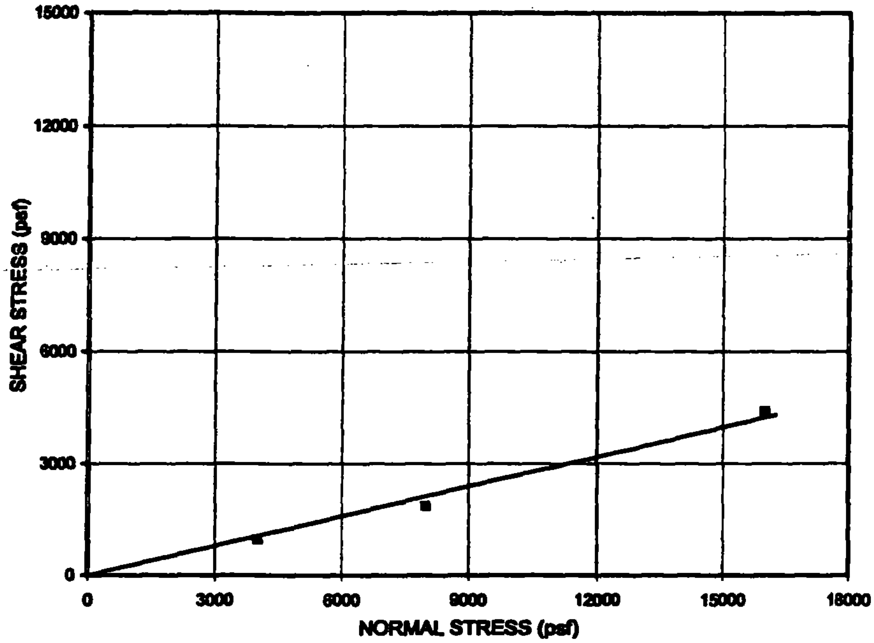
Adhesion: 0 psf

Friction Angle: 15 degrees

Coefficient of Friction: 0.26

Note: Intercept Adjusted to "0".

NOTE: GRAPH NOT TO SCALE



STRENGTH ENVELOPE

(at 2.5 in. displacement)

Test Point	Normal Stress psi	Normal Stress psf	Shear Stress psf	Secant Friction Angle
1.	27.8	4000	610	9
2.	55.6	8000	1300	9
3.	111.1	16000	2960	10

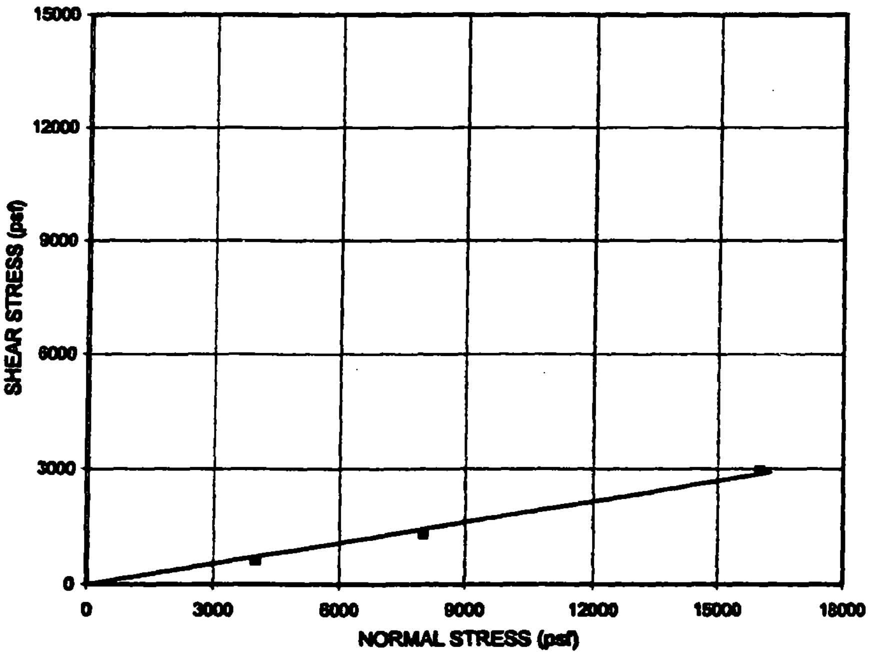
Adhesion: 0 psf

Friction Angle: 10 degrees

Coefficient of Friction: 0.18

Note: Intercept Adjusted to "0".

NOTE: GRAPH NOT TO SCALE



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DCN: LSDS-rp (rev., 03/01/04)

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LABORATORY SERVICES

LARGE SCALE DIRECT SHEAR REPORT

Test Method D-5321A

Report Date: April 6, 2007
Project No: 061204.05

Client Name: ALLIED WASTE INC. Project Name: WASATCH PHASE 2A

Superstrata: ⇦ Drainage Layer

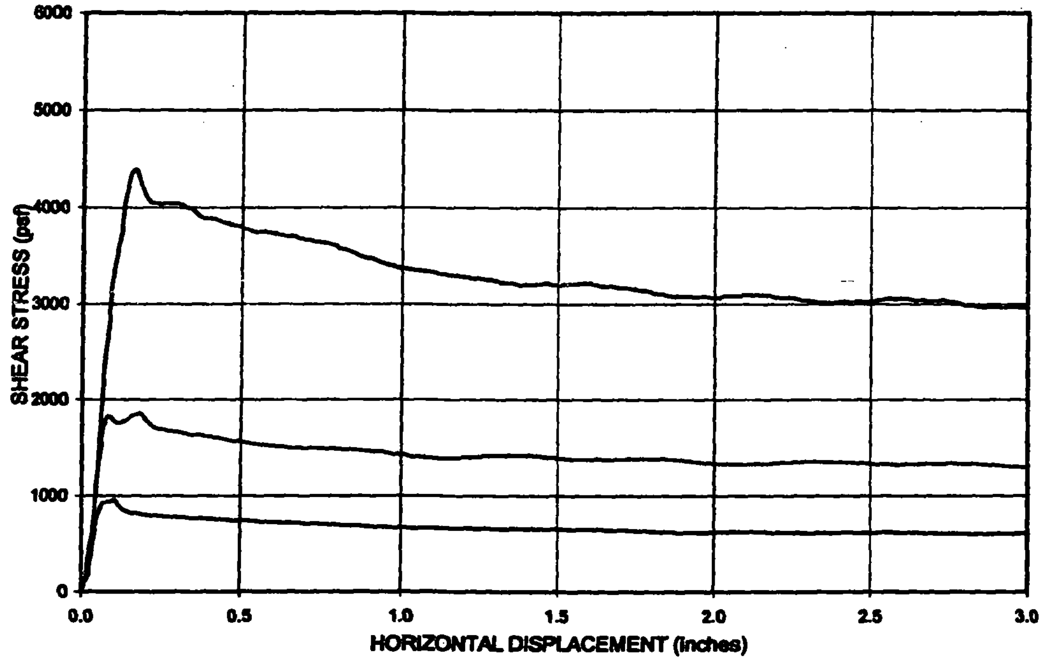
Material 1: ⇦ GSE 60 mil HDPE Smooth, Roll# 108120131 LSN: ALD Clamped

Material 2: ⇨ GSE Single side textile Geocomposite, Roll# 131238484 LSN: ALG Clamped

Substrata: ⇨ Concrete Board

DISPLACEMENT vs. SHEAR STRESS

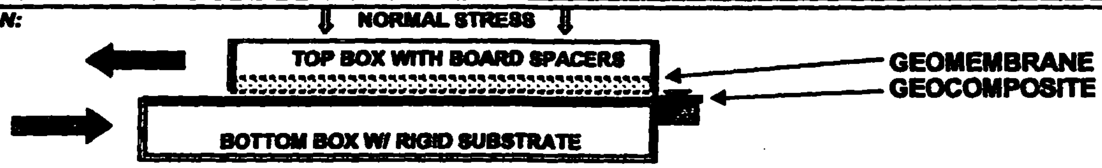
Test Point	Normal Stress psi	Normal Stress psf
1.	27.8	4000
2.	55.6	8000
3.	111.1	16000



STANDARD CONDITIONS: **SHEAR DISPLACEMENT RATE: 0.04 in/min**

1. The "gap" between shear boxes was set at 80 mil (2.0 mm)
2. The test specimens were flooded during testing unless otherwise noted.
3. High Normal Stresses, >5psi (35 kPa) was applied using air pressure.
4. Low Normal Stresses, <5psi (35 kPa) was applied using dead weights.
5. The tests were terminated after 3.0"(75 mm) of displacement unless otherwise noted.
6. Tests were performed in general accordance with ASTM procedure D-5321 using a Brainard-Kilman LG-112 direct shear machine with an effective area of 12" x 12" (300 x300 mm).

TEST ORIENTATION:



SPECIAL TEST NOTES:

1. Each specimen of geocomposite was cut to 14" x 20" and clamped to the lower shear box.
2. Each specimen of geomembrane was cut to 12" x 12" and clamped to the upper shear box.
3. Each test specimen was consolidated for 1 hour at the specified normal stress, then sheared.
4. The test was performed in a "wet" or "flooded" condition.
5. Shearing occurred at the interface of the geocomposite and geomembrane specimens.
6. The Friction Angle and Adhesion (or Cohesion) results given here are based on a mathematically determined best fit line.
7. Further interpretation should be conducted by a qualified professional experienced in geosynthetic and geotechnical engineering.

Friction Angle and Cohesion												
	HDPE Textured / GCL		HDPE Smooth / GCL		HDPE Textured / Single Sided Geocomposite HDPE Textured / Geonet		HDPE Textured / Double Sided Geocomposite HDPE Textured / Geotextile		HDPE Smooth / Single Sided Geocomposite HDPE Smooth / Geonet		HDPE Smooth / Double Sided Geocomposite HDPE Smooth / Geotextile	
	Friction Angle	Cohesion	Friction Angle	Cohesion	Friction Angle	Cohesion	Friction Angle	Cohesion	Friction Angle	Cohesion	Friction Angle	Cohesion
	13	0	29	20	11	60	21	90	13	0	12	20
	10	290	13	390	19	30	16	80	11	0	9	0
	8	460	12	360	9	40	15	90	14	0	10	20
	7	460	6	300	15	20	12	230	10	70	10	20
	8	410	10	70	14	0	16	110	10	30	9	30
	9	280	11	0	16	160	21	90			8	50
	5	480	11	270	23	20	16	80			10	20
	16	90	9	50	16	0	26	10			9	20
	2	570	8	310	19	49	17	40			9	20
	16	120			12	0	21	50			9	20
	11	20			15	0	14	160			10	0
	11	290			15	24	13	490			10	0
	29	0			15	86	14	250			11	0
	10	70			12	410	13	470			11	0
	11	90			14	0	11	520			9	70
	12	100					11	220			8	20
	11	100					20	100			9	80
	11	110					15	130			8	40
	13	60					12	130			8	0
	14	40					14	120			9	0
	12	110					13	160				
	7	330					16	720				
	33	70					13	350				
	20	30					16	680				
	13	70					11	180				
	12	310					14	300				
	12	350					15	250				
	13	280					12	760				
	25	110					13	340				
	18	500					22	90				
	16	190					11	440				
	17	220					11	450				
	11	940					11	520				
	22	230					11	350				
	12	370					13	160				
	28	190					12	390				
	34	220					15	140				
	15	50					12	590				
	11	220					9	810				
							10	450				
							11	390				
							13	390				
							14	290				
							12	310				
Count	39	39	9	9	15	15	44	44	5	5	20	20
Maximum	34	940	29	390	23	410	26	810	14	70	12	80
Minimum	2	0	6	0	9	0	9	10	10	0	8	0
Average	14	226	12	197	15	60	14	295	12	20	9	22
Standad Deviation	16	491	12	195	7	221	9	405	2	36	2	41
Max - Average	20	714	17	193	8	350	12	515	2	50	3	59
Average - Min	12	226	6	197	6	60	5	285	2	20	1	22

APPENDIX B
SEISMIC HAZARD DATA

Faults Near Wasatch Regional Landfill																	
Project: Wasatch Regional Landfill														Jul-08			
FAULT NAME	u (dyne/cm2)	Fault Length (km)	Fault Width (km)	A (km2)	A (cm2)	S (mm/yr)	S (cm/yr)	uAS	M(char)	Mmin	b (assumed)	β	c2	Rrup	Rjb	Rx	Idriss
East Great Salt Lake fault zone, Antelope Island section	3.00E+11	35	15	525	5.25E+12	0.8	0.08	1.3E+24	6.7	5	1	2.30	0.39	45	45	0	
Stansbury fault zone	3.00E+11	50	15	750	7.5E+12	0.2	0.02	4.5E+23	6.9	5	1	2.30	0.27	14	14	0	0.436
Lakeside Mountains (west side) fault	3.00E+11	5	15	75	7.5E+11	0.2	0.02	4.5E+22	5.9	5	1	2.30	3.25	10	10	0	0.273
Skull Valley (mid-valley) faults	3.00E+11	55	15	825	8.25E+12	0.2	0.02	5.0E+23	6.9	5	1	2.30	0.25	35	35	0	0.182
Puddle Valley fault zone	3.00E+11	7	15	105	1.05E+12	0.2	0.02	6.3E+22	6.1	5	1	2.30	2.16	24	24	0	0.136
Oquirrh fault zone	3.00E+11	21	50	1050	1.05E+13	0.8	0.08	2.5E+24	7.0	5	1	2.30	0.19	47	47	0	0.135
East Great Salt Lake fault zone, Promontory section	3.00E+11	37	15	555	5.55E+12	0.8	0.08	1.3E+24	6.8	5	1	2.30	0.37	48	48	0	0.121
East Great Salt Lake fault zone, Antelope Island section	3.00E+11	26	15	390	3.9E+12	0.8	0.08	9.4E+23	6.6	5	1	2.30	0.53	40	40	0	0.11
Southern Oquirrh Mountains fault zone	3.00E+11	24	50	1200	1.2E+13	0.8	0.08	2.9E+24	7.1	5	1	2.30	0.17	58	58	0	0.109
East Great Salt Lake fault zone, Fremont Island section	3.00E+11	13	15	195	1.95E+12	0.8	0.08	4.7E+23	6.3	5	1	2.30	1.09	40	40	0	0.086
Wasatch fault zone, Salt Lake City section	3.00E+11	23	50	1150	1.15E+13	4	0.4	1.4E+25	7.1	5	1	2.30	0.18	72	72	0	0.083
Wasatch fault zone, Weber section	3.00E+11	20	50	1000	1E+13	4	0.4	1.2E+25	7.0	5	1	2.30	0.20	72	72	0	0.079
Wasatch fault zone, Clarkston Mountain section	3.00E+11	43	50	2150	2.15E+13	0.2	0.02	1.3E+24	7.3	5	1	2.30	0.10	80	80	0	0.079
Wasatch fault zone, Provo section	3.00E+11	23	50	1150	1.15E+13	4	0.4	1.4E+25	7.1	5	1	2.30	0.18	80	80	0	0.072
West Valley fault zone, Taylorsville fault	3.00E+11	31	15	465	4.65E+12	0.2	0.02	2.8E+23	6.7	5	1	2.30	0.44	64	64	0	0.066
West Valley fault zone, Granger section	3.00E+11	32	15	480	4.8E+12	0.8	0.08	1.2E+24	6.7	5	1	2.30	0.43	64	64	0	0.066
Antelope Range-Kingsley Mountains fault zone	3.00E+11	93	15	1395	1.395E+13	0.2	0.02	8.4E+23	7.2	5	1	2.30	0.15	105	105	0	0.052
Wasatch fault zone, Nephi section	3.00E+11	56	50	2800	2.8E+13	4	0.4	3.4E+25	7.4	5	1	2.30	0.07	113	113	0	0.052
Wasatch fault zone, Brigham City section	3.00E+11	15	50	750	7.5E+12	4	0.4	9.0E+24	6.9	5	1	2.30	0.27	97	97	0	0.05
Wasatch fault zone, Collinston section	3.00E+11	35	50	1750	1.75E+13	0.2	0.02	1.1E+24	7.2	5	1	2.30	0.12	113	113	0	0.047
Porcupine Mountain faults	3.00E+11	59	15	885	8.85E+12	0.2	0.02	5.3E+23	7.0	5	1	2.30	0.23	105	105	0	0.047
West Valley fault zone, Granger section	3.00E+11	16	15	240	2.4E+12	0.8	0.08	5.8E+23	6.4	5	1	2.30	0.87	72	72	0	0.043
Topliff Hill fault zone	3.00E+11	20	15	300	3E+12	0.2	0.02	1.8E+23	6.5	5	1	2.30	0.69	80	80	0	0.04
North Promontory fault	3.00E+11	30	15	450	4.5E+12	0.2	0.02	2.7E+23	6.7	5	1	2.30	0.45	97	97	0	0.037
Broadmouth Canyon faults	3.00E+11	35	15	525	5.25E+12	0.2	0.02	3.2E+23	6.7	5	1	2.30	0.39	97	97	0	0.037
Saint John Station fault zone	3.00E+11	5	15	75	7.5E+11	0.2	0.02	4.5E+22	5.9	5	1	2.30	3.25	60	60	0	0.036
Sheeprock fault zone	3.00E+11	12	15	180	1.8E+12	0.2	0.02	1.1E+23	6.3	5	1	2.30	1.18	80	80	0	0.034
Drum Mountains fault zone	3.00E+11	52	15	780	7.8E+12	0.2	0.02	4.7E+23	6.9	5	1	2.30	0.26	129	129	0	0.033
Clover fault zone	3.00E+11	4	15	60	6E+11	0.2	0.02	3.6E+22	5.8	5	1	2.30	4.34	61	61	0	0.032
Wasatch fault zone, Levan segment	3.00E+11	43	50	2150	2.15E+13	4	0.4	2.6E+25	7.3	5	1	2.30	0.10	153	153	0	0.031
Utah Lake faults	3.00E+11	10	15	150	1.5E+12	0.2	0.02	9.0E+22	6.2	5	1	2.30	1.45	80	80	0	0.031
West Cache fault zone, Clarkston fault	3.00E+11	49	15	735	7.35E+12	0.8	0.08	1.8E+24	6.9	5	1	2.30	0.28	137	137	0	0.03
East Cache fault zone, southern section	3.00E+11	35	15	525	5.25E+12	0.2	0.02	3.2E+23	6.7	5	1	2.30	0.39	113	113	0	0.029
Wasatch fault zone, Fayette section	3.00E+11	59	50	2950	2.95E+13	0.2	0.02	1.8E+24	7.5	5	1	2.30	0.07	177	177	0	0.028
Strawberry fault	3.00E+11	41	15	615	6.15E+12	0.2	0.02	3.7E+23	6.8	5	1	2.30	0.33	137	137	0	0.028
Antelope Range-Kingsley Mountains fault zone	3.00E+11	65	15	975	9.75E+12	0.2	0.02	5.9E+23	7.0	5	1	2.30	0.21	148	148	0	0.028
James Peak fault	3.00E+11	20	15	300	3E+12	0.2	0.02	1.8E+23	6.5	5	1	2.30	0.69	105	105	0	0.027
West Cache fault zone, Wellsville fault	3.00E+11	35	15	525	5.25E+12	0.2	0.02	3.2E+23	6.7	5	1	2.30	0.39	121	121	0	0.026
Fish Springs fault	3.00E+11	30	15	450	4.5E+12	0.2	0.02	2.7E+23	6.7	5	1	2.30	0.45	121	121	0	0.026
Gunnison fault	3.00E+11	59	15	885	8.85E+12	4	0.4	1.1E+25	7.0	5	1	2.30	0.23	158	158	0	0.025
House Range (west side) fault	3.00E+11	46	15	690	6.9E+12	0.2	0.02	4.1E+23	6.9	5	1	2.30	0.30	153	153	0	0.025
West Cache fault zone, Junction Hills fault	3.00E+11	30	15	450	4.5E+12	0.2	0.02	2.7E+23	6.7	5	1	2.30	0.45	129	129	0	0.024
Hansel Valley fault	3.00E+11	10	15	150	1.5E+12	0.2	0.02	9.0E+22	6.2	5	1	2.30	1.46	97	97	0	0.023
Schell Creek Range fault	3.00E+11	99	15	1485	1.485E+13	0.2	0.02	8.9E+23	7.2	5	1	2.30	0.14	177	177	0	0.023
Independence Valley fault zone, southern section	3.00E+11	43	15	645	6.45E+12	0.2	0.02	3.9E+23	6.8	5	1	2.30	0.32	161	161	0	0.022
Pilot Creek Valley fault	3.00E+11	27	15	405	4.05E+12	0.2	0.02	2.4E+23	6.6	5	1	2.30	0.51	127	127	0	0.022
East Dayton-Oxford fault	3.00E+11	24	15	360	3.6E+12	0.2	0.02	2.2E+23	6.6	5	1	2.30	0.57	129	129	0	0.022
Vernon Hills fault zone	3.00E+11	4	15	60	6E+11	0.2	0.02	3.6E+22	5.8	5	1	2.30	4.34	84	84	0	0.019
Porcupine Mountain faults	3.00E+11	16	15	240	2.4E+12	0.2	0.02	1.4E+23	6.4	5	1	2.30	0.87	129	129	0	0.018
Deep Creek Range (northwest side) fault zone	3.00E+11	11	15	165	1.65E+12	0.2	0.02	9.9E+22	6.2	5	1	2.30	1.30	121	121	0	0.016
Morgan fault, central section	3.00E+11	3	15	45	4.5E+11	0.2	0.02	2.7E+22	5.7	5	1	2.30	6.56	88	88	0	0.016
Crater Bench faults	3.00E+11	16	15	240	2.4E+12	0.2	0.02	1.4E+23	6.4	5	1	2.30	0.87	137	137	0	0.016
Faults on eastern side of Curlew Valley	3.00E+11	13	15	195	1.95E+12	0.2	0.02	1.2E+23	6.3	5	1	2.30	1.09	132	132	0	0.015
Martin Ranch fault	3.00E+11	26	15	390	3.9E+12	0.8	0.08	9.4E+23	6.6	5	1	2.30	0.53	161	161	0	0.015
Crawford Mountains (west side) fault	3.00E+11	22	15	330	3.3E+12	0.2	0.02	2.0E+23	6.5	5	1	2.30	0.63	153	153	0	0.015
Spruce Mountain Ridge fault zone	3.00E+11	31	15	465	4.65E+12	0.2	0.02	2.8E+23	6.7	5	1	2.30	0.44	175	175	0	0.014
Clear Lake fault zone	3.00E+11	36	15	540	5.4E+12	0.2	0.02	3.2E+23	6.7	5	1	2.30	0.38	177	177	0	0.014
Silver Island Mountains (southeast side) fault	3.00E+11	2	15	30	3E+11	0.2	0.02	1.8E+22	5.5	5	1	2.30	13.52	92	92	0	0.013
Bear River fault zone	3.00E+11	20	15	300	3E+12	0.2	0.02	1.8E+23	6.5	5	1	2.30	0.69	161	161	0	0.013
Whitney Canyon fault	3.00E+11	17	15	255	2.55E+12	0.2	0.02	1.5E+23	6.4	5	1	2.30	0.82	153	153	0	0.013
Little Valley faults	3.00E+11	19	15	285	2.85E+12	0.2	0.02	1.7E+23	6.5	5	1	2.30	0.73	161	161	0	0.013
East Cache fault zone, central section	3.00E+11	5	15	75	7.5E+11	0.8	0.08	1.8E+23	5.9	5	1	2.30	3.25	121	121	0	0.012
Southern Spring Valley fault zone	3.00E+11	40	15	600	6E+12	0.2	0.02	3.6E+23	6.8	5	1	2.30	0.34	225	225	0	0.012
Snow Lake graben	3.00E+11	25	15	375	3.75E+12	0.2	0.02	2.3E+23	6.6	5	1	2.30	0.55	193	193	0	0.011
Eastern Bear Lake fault, southern section	3.00E+11	6	15	90	9E+11	0.8	0.08	2.2E+23	6.0	5	1	2.30	2.60	161	161	0	0.008
Scipio Valley faults	3.00E+11	7	15	105	1.05E+12	0.2	0.02	6.3E+22	6.1	5	1	2.30	2.16	169	169	0	0.008
Scipio faults	3.00E+11	13	15	195	1.95E+12	0.2	0.02	1.2E+23	6.3	5	1	2.30	1.09	185	185	0	0.008
Pavant Range fault	3.00E+11	14	15	210	2.1E+12	0.2	0.02	1.3E+23	6.3	5	1	2.30	1.00	193	193	0	0.008
Southern Snake Range fault zone	3.00E+11	28	15	420	4.2E+12	0.2	0.02	2.5E+23	6.6	5	1	2.30	0.49	225	225	0	0.008
Unnamed fault on west side of Snake Range	3.00E+11	26	15	390	3.9E+12	0.2	0.02	2.3E+23	6.6	5	1	2.30	0.53	241	241	0	0.007
Stinking Springs fault	3.00E+11	4	15	60	6E+11	0.2	0.02	3.6E+22	5.8	5	1	2.30	4.34	161	161	0	0.006
Rock Creek fault	3.00E+11	6	15	90	9E+11	0.8	0.08	2.2E+23	6.0	5	1	2.30	2.60	209	209	0	0.005
Sugarville area faults	3.00E+11	4	15	60	6E+11	0.2	0.02	3.6E+22	5.8	5	1	2.30	4.34	185	185	0	0.005

Earthquake Search Results

Circle Search Earthquakes = 24

Radius: 165 km

Date Range: 1000 - 2007

Magnitude Range: 4.5 - 9.0

Circle Center Point Latitude: 40.852N   Longutude: -112.749W

Note:
Type of Magnitude *UK is assumed to be ML* based on occurrence time

	CAT	YEAR	MO	DAY	ORIG TIME	LAT	LONG	DEPTH (km)	MAGNITUDE	DIST (km)	TYPE OF MAGNITUDE	Mw
1	SRA	1934	3	12	150540	41.5	-112.5		6.6	74	UKSRA	6.8
2	SRA	1934	3	12	1729	41.5	-112.5		4.8	74	MLSRA	4.8
3	SRA	1934	3	12	1812	41.5	-112.5		5.1	74	MLSRA	5.1
4	SRA	1934	3	12	182013	41.5	-112.5		6	74	UKSRA	6.1
5	SRA	1934	3	15	1201	41.5	-112.5		5.1	74	MLSRA	5.1
6	SRA	1934	3	15	1346	41.5	-112.5		4.8	74	MLSRA	4.8
7	SRA	1934	4	7	216	41.5	-111.5		5.5	127	MLSRA	5.5
8	SRA	1934	4	14	212632	41.5	-112.5		5.3	74	UKSRA	5.3
9	SRA	1934	5	6	80949	41.5	-113		5.5	74	UKSRA	5.5
10	SRA	1962	8	30	133524.4	42.02	-111.74	7	5.7	158	MLSRA	5.7
11	SRA	1962	9	5	160427.8	40.72	-112.09	7	5.2	57	MLSRA	5.2
12	SRA	1963	7	17	192039.6	39.53	-111.91	7	4.9	163	mb gs	4.9
13	SRA	1966	3	17	114747.4	41.66	-111.56	7	4.6	134	MLSRA	4.6
14	SRA	1970	3	29	124040.3	41.66	-113.84	7	4.7	128	MLSRA	4.7
15	SRA	1972	3	6	133324.9	41.88	-111.61	7	4.6	148	mb gs	4.6
16	SRA	1972	10	1	194229.5	40.51	-111.35	7	4.7	124	mb gs	4.7
17	SRA	1975	3	28	23106	42.06	-112.52	5	6.1	135	mb gs	6.8
18	SRA	1975	3	29	130119.9	42.03	-112.52	7	4.7	132	mb gs	4.7
19	SRA	1975	4	2	210646.2	42.09	-112.44	6	4.7	139	mb gs	4.7
20	SRA	1975	4	7	134234.6	42.05	-112.49	6	4.6	134	mb gs	4.6
21	SRA	1978	11	30	65340.1	42.11	-112.49	4	4.7	141	MLSRA	4.7
22	SRA	1980	5	24	100336.3	39.94	-111.97	5	5	120	mb gs	5
23	SRA	1981	2	20	91301.2	40.32	-111.74	1	4.7	103	mb gs	4.7
24	SRA	1983	10	8	115753.8	40.75	-111.99	6	4.5	64	mb gs	4.5

APPENDIX C
DISPLACEMENT ANALYSES

SIMPLIFIED SEISMIC DESIGN PROCEDURE FOR GEOSYNTHETIC-LINED, SOLID-WASTE LANDFILLS

This analysis is based on the paper "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills," A Technical Paper by J.D. Bray, E.M. Rathje, A.J. Augello, and S.M. Merry, published in Geosynthetics International 1998, Vol. 5, Nos. 1-2, Pages 203-235

Base Sliding

Description	Value & Source	
Name of Landfill	Wasatch Regional Landfill	
Section Details	A-A' **Option 1**	
Fault & Earthquake Description & Parameters:		
Near-field fault considered	Stansbury Fault	
Magnitude of Earthquake (M_w) - with 10% or 2% probability of exceedance in 50 years (as locally required)	6.9	*USGS*
Epicentral Distance from site	14 miles 22.58 km	*USGS*
Estimated Max. Horiz. Accel. (MHA_{Rock})	0.27 .g	*Bray-Fig. 2a*
Mean Time Period of Earthquake (T_m)	0.53 sec	*Rathje et al., 1998; Bray Fig. 2b*
Significant Duration (D_{5-95})	16 sec	*Abrahamson/Silva, 1996; Bray Fig. 2c*
Horiz. Earthquake Coeff. For pseudostatic stabiltiy analysis (k)	0.436	*Vector Analyses*
Screening for Displacement Analysis		
Yield Accel. Coeff. for Base Sliding (k_y)	0.123	*Vector Analyses*
Acceptable Displacement at the base due to Sliding:	300 mm	*Common Practice*
Screening Logic: *Is $k > k_y$?*	Yes	
Screening Result	**Displacements in excess of 300mm at the base is expected; Displacement Analysis is advised.**	

Base Sliding - Permanent Displacement Calculations		
Max. Height of Proposed Landfill (H)	300 ft 91.5 m	*As Designed*
Shear Wave Velocity - Top third (V_T)	200 m/sec	*Kavazanjian et al. 1996; Bray-Fig. 3*
- Middle third (V_M)	310 m/sec	
- Bottom third (V_B)	340 m/sec	
Avergage Shear Wave Velocity ($V_{s\text{-}avg}$)	283 m/sec	*$=V_T+V_M+V_B/3$*
Fundamental Period of Landfill (T_s)	1.3 sec	*$= 4H/V_{s\text{-}avg}$*
Time Period Ratio - T_s/T_m	2.4	
Nonlinear Response Factor of Waste (NRF = MHA_{Site}/MHA_{Rock})	1.12	*= 0.6225+0.9196 * EXP(-MHA_{Rock}/ /g/0.4449)*
Max. Horiz. Accel. for the Site (MHA_{Site})	0.30 .g	*= NRF * MHA_{Rock}*
For 16% Probability of Exceedance -		
Normalized Maximum Horizontal Equivalent Acceleration ($MHEA_{Norm}$)	0.38 .g	*Bray-Fig. 6; =$MHEA_{Base}/MHA_{Site}$*
Maximum Horizontal Equivalent Acceleration ($MHEA_{Base}$)	0.12 .g	*= $MHEA_{Norm}$ *MHA_{Site}*
Max. Seismic Accel. Coefficient (k_{max})	0.12	*$k_{max}=MHEA_{Base}/g$*
Acceleration Ratio (k_y/k_{max})	1.07	
Normalized Sliding Displacemnt. (U_{Norm})	0.6 mm/s	*Bray-Fig. 11*
Permanent Displacement (U) - @ probability of 16% Exceedance	1.11 mm **0.04 inch**	*U= U_{Norm} * $D_{5\text{-}95}$ * k_{max}*
For 50% Probability of Exceedance -		
Normalized Maximum Horizontal Equivalent Acceleration ($MHEA_{Norm}$)	0.27	*Bray Fig. 6; = EXP(-0.624-0.7831*ln(T_s/T_m))*
Maximum Horizontal Equivalent Acceleration ($MHEA_{Base}$)	0.08 .g	*= $MHEA_{Norm}$ *MHA_{Site}*
Max. Seismic Accel. Coefficient (k_{max})	0.08	*$k_{max}=MHEA_{Base}/g$*
Acceleration Ratio (k_y/k_{max})	1.51	
Normalized Sliding Displacemnt. (U_{Norm})	0.00 mm/s	*Bray-Fig. 11; =10^(1.87-3.477 *k_y/k_{max})*
Permanent Displacement (U) - @ probability of 50% Exceedance	0.01 mm **0.00 inch**	*U= U_{Norm} * $D_{5\text{-}95}$ * k_{max}*

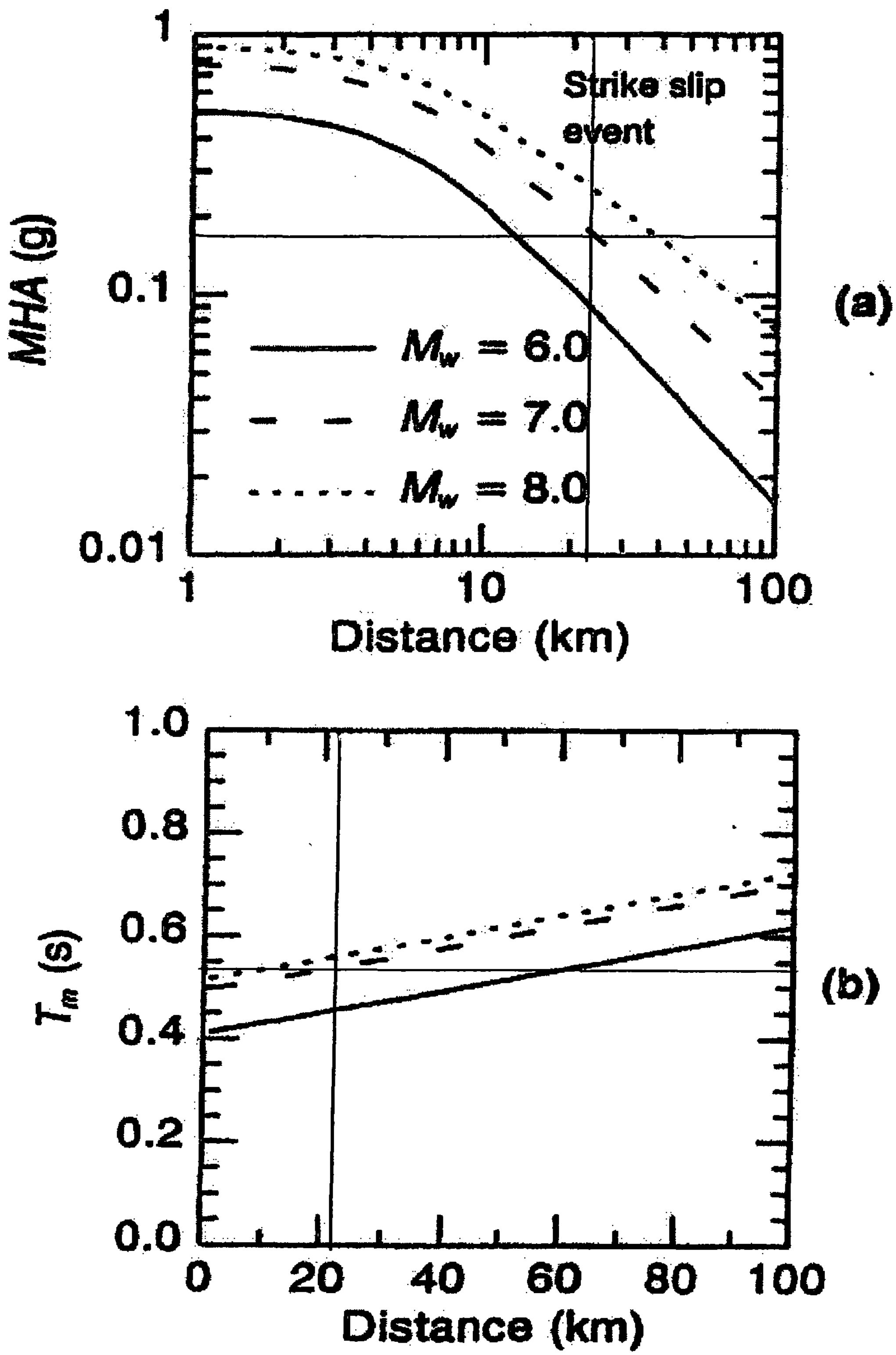
SIMPLIFIED SEISMIC DESIGN PROCEDURE FOR GEOSYNTHETIC-LINED, SOLID-WASTE LANDFILLS

This analysis is based on the paper "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills," A Technical Paper by J.D. Bray, E.M. Rathje, A.J. Augello, and S.M. Merry, published in Geosynthetics International 1998, Vol. 5, Nos. 1-2, Pages 203-235

Base Sliding

Description	Value & Source	
Name of Landfill	Wasatch Regional Landfill	
Section Details	A-A' **Option 2**	
Fault & Earthquake Description & Parameters:		
Near-field fault considered	Stansbury Fault	
Magnitude of Earthquake (M_w) - with 10% or 2% probability of exceedance in 50 years (as locally required)	6.9	*USGS*
Epicentral Distance from site	14 miles 22.58 km	*USGS*
Estimated Max. Horiz. Accel. (MHA_{Rock})	0.27 .g	*Bray-Fig. 2a*
Mean Time Period of Earthquake (T_m)	0.53 sec	*Rathje et al., 1998; Bray Fig. 2b*
Significant Duration (D_{5-95})	16 sec	*Abrahamson/Silva, 1996; Bray Fig. 2c*
Horiz. Earthquake Coeff. For pseudostatic stabiltiy analysis (k)	0.436	*Vector Analyses*
Screening for Displacement Analysis		
Yield Accel. Coeff. for Base Sliding (k_y)	0.175	*Vector Analyses*
Acceptable Displacement at the base due to Sliding:	300 mm	*Common Practice*
Screening Logic: *Is $k > k_y$?*	Yes	
Screening Result	**Displacements in excess of 300mm at the base is expected; Displacement Analysis is advised.**	

Base Sliding - Permanent Displacement Calculations		
Max. Height of Proposed Landfill (H)	300 ft 91.5 m	*As Designed*
Shear Wave Velocity - Top third (V_T)	200 m/sec	*Kavazanjian et al. 1996; Bray-Fig. 3*
- Middle third (V_M)	310 m/sec	
- Bottom third (V_B)	340 m/sec	
Avergage Shear Wave Velocity ($V_{s\text{-}avg}$)	283 m/sec	$=V_T+V_M+V_B/3$
Fundamental Period of Landfill (T_s)	1.3 sec	$= 4H/V_{s\text{-}avg}$
Time Period Ratio - T_s/T_m	2.4	
Nonlinear Response Factor of Waste (NRF = MHA_{Site}/MHA_{Rock})	1.12	$= 0.6225+0.9196 * EXP(-MHA_{Rock}/ /g/0.4449)$
Max. Horiz. Accel. for the Site (MHA_{Site})	0.30 .g	$= NRF * MHA_{Rock}$
For 16% Probability of Exceedance -		
Normalized Maximum Horizontal Equivalent Acceleration ($MHEA_{Norm}$)	0.38 .g	*Bray-Fig. 6;* $=MHEA_{Base}/MHA_{Site}$
Maximum Horizontal Equivalent Acceleration ($MHEA_{Base}$)	0.12 .g	$= MHEA_{Norm} * MHA_{Site}$
Max. Seismic Accel. Coefficient (k_{max})	0.12	$k_{max} = MHEA_{Base/} g$
Acceleration Ratio (k_y/k_{max})	1.52	
Normalized Sliding Displacemnt. (U_{Norm})	0.6 mm/s	*Bray-Fig. 11*
Permanent Displacement (U) - @ probability of 16% Exceedance	1.11 mm **0.04 inch**	$U= U_{Norm} * D_{5\text{-}95} * k_{max}$
For 50% Probability of Exceedance -		
Normalized Maximum Horizontal Equivalent Acceleration ($MHEA_{Norm}$)	0.30	*Bray Fig. 6;* $= EXP(-0.624- 0.7831*ln(T_s/T_m))$
Maximum Horizontal Equivalent Acceleration ($MHEA_{Base}$)	0.09 .g	$= MHEA_{Norm} * MHA_{Site}$
Max. Seismic Accel. Coefficient (k_{max})	0.09	$k_{max} = MHEA_{Base/} g$
Acceleration Ratio (k_y/k_{max})	1.92	
Normalized Sliding Displacemnt. (U_{Norm})	0.00 mm/s	*Bray-Fig. 11;* $=10^{\wedge}(1.87-3.477 *k_y/k_{max})$
Permanent Displacement (U) - @ probability of 50% Exceedance	0.00 mm **0.00 inch**	$U= U_{Norm} * D_{5\text{-}95} * k_{max}$



1
0.1
0.01
MHA (g)
Strike slip
event
M_w = 6.0
M_w = 7.0
M_w = 8.0
1
10
100
Distance (km)
(a)
1.24
1.0
0.8
0.6
0.4
0.2
0.0
T_m (s)
0
20
40
60
80
100
Distance (km)
(b)

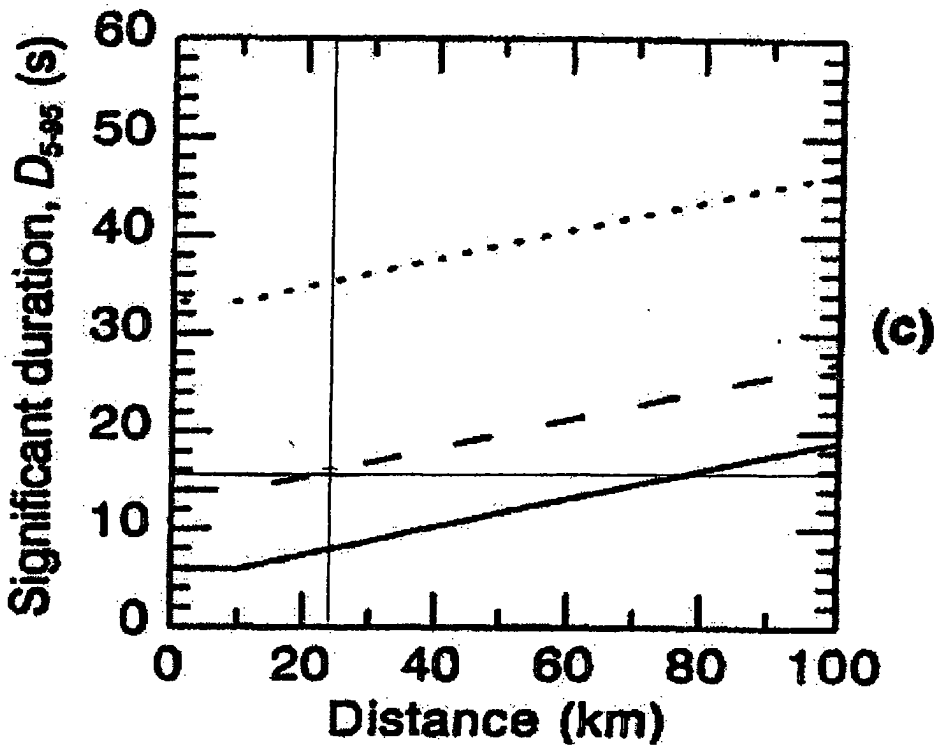


Figure 2. Simplified Characterization of earthquake rock motions: (a) intensity, MHA for strike-slip faults (for reverse faults, use 1.3xMHA for Mw ≥ 6.4 & 1.64xMHA for Mw = 6.0, with linear interpolation for 6.0 < Mw < 6.4) (Abrahamson & Silva, 1997); (b) frequency content, Tm (Rathje et al., 1998); (c) duration, D_{5-95} (Abrahamson & Silva, 1996).

1.42
0.06

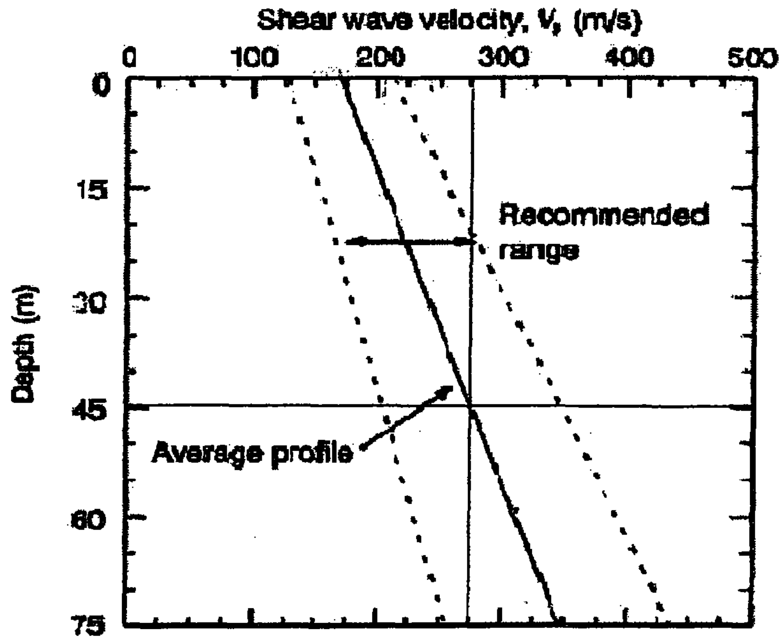


Figure 3. Shear wave velocity profiles for municipal solid-waste (after Kavazanjian et al., 1996)

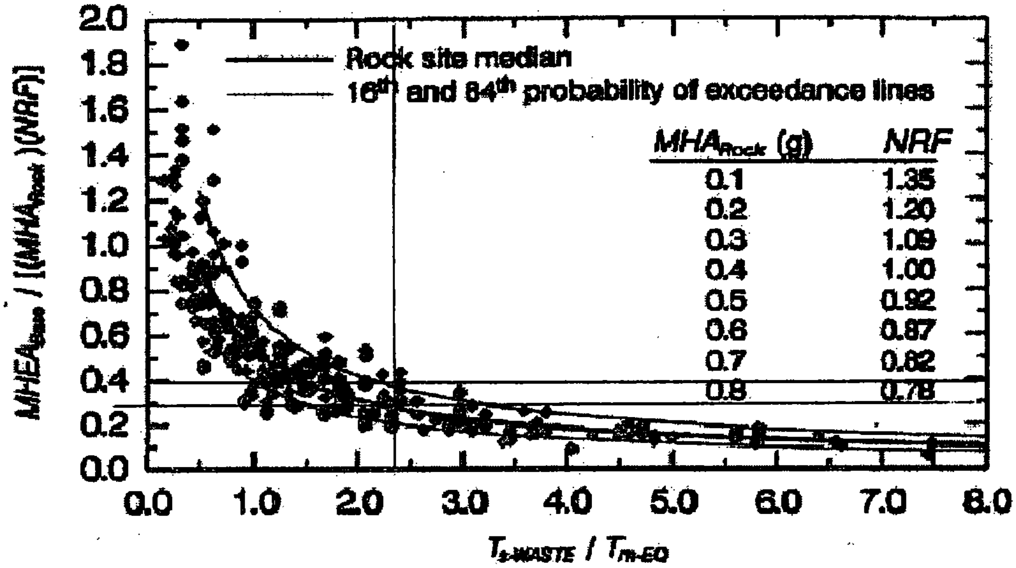


Figure 6. Normalized maximum horizontal equivalent acceleration for base sliding versus normalized fundamental period of waste fill (adapted from Bray & Rathje, 1998).

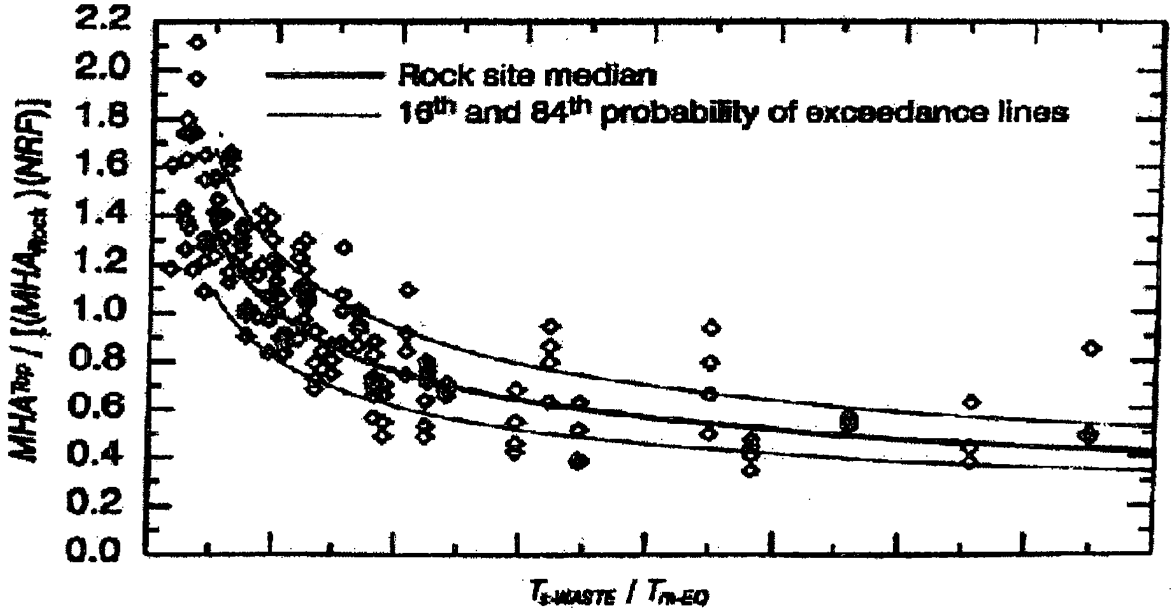


Figure 8. Normalized maximum horizontal acceleration at the top versus normalized fundamental period of waste fill (adapted from Bray & Rathje, 1998).

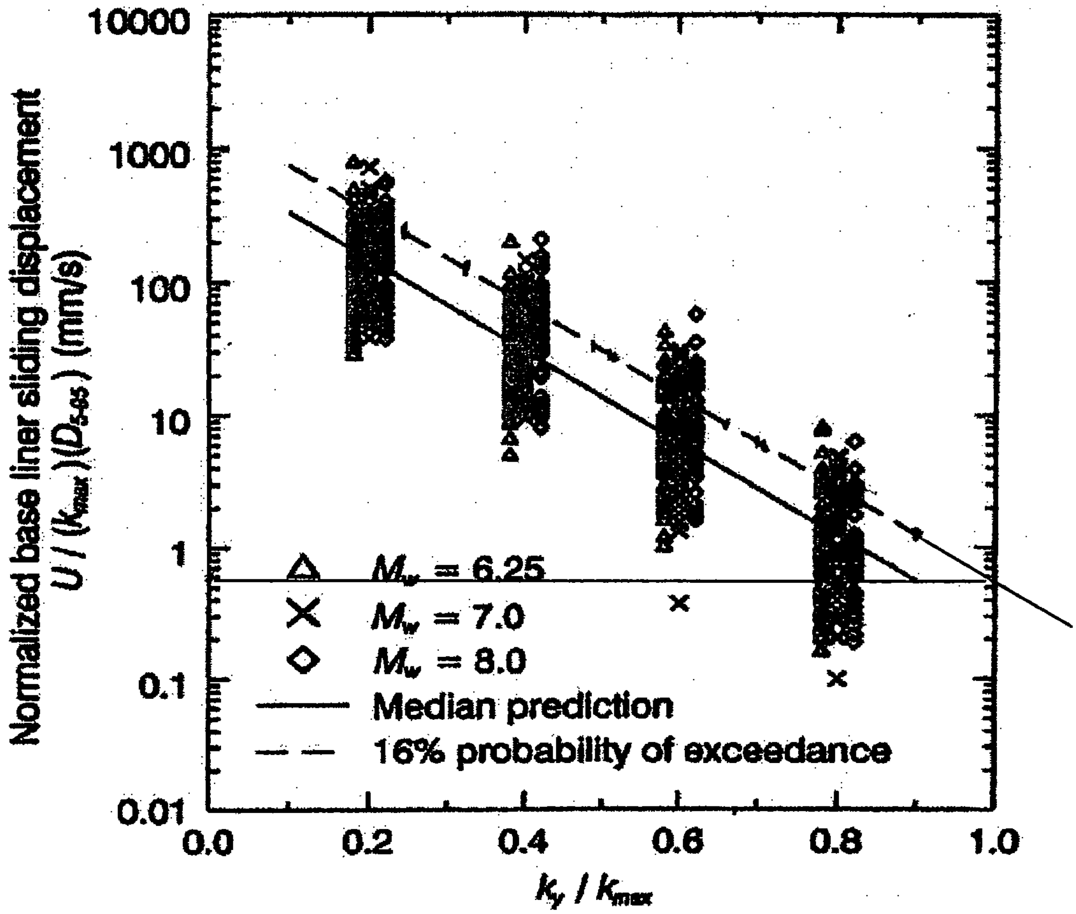


Figure 11. Normalized base liner sliding displacements
(from Bray & Rathje, 1998)

References:

Abrahamson, N.A. and Silva, W.J., 1996; "Empirical Ground Motion Models," Report prepared for Brrokheaven National Laboratory, New York, New York, 144p.

ASCE, 2002; "Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Landslide Hazards in California," A document published by the Southern California Earthquake Center.

Bray, J.D., Rathje, E.M., Augello, A.J., and Merry, S.M., 1998; "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills," Geosynthetics International, Vol. 5, Nos. 1-2, Pages 203-235.

Kavajanjian, Jr., E., Matasovic, N., Stokoe, K.H., and Bray, J.D., 1996; "In Situ Shear Wave Velocity of Solid Waste from Surface Wave Measurements," Proceedings of the Second International Geotechnics, Balkema, Vol. 1, Osaka, Japan, pp. 97-102

Rathje, E.M., Abrahamson, N., and Bray, J.D., 1998; "Simplified Frequency Content Estimates of Earthquake Ground Motions," Journal of Geotechnical Engineering, Vol. 124, No. 2, pp. 150-159.

APPENDIX D
STABILITY ANALYSES RESULTS

Infinite Slope Method of Cover Slope Stability Analysis
Thiel and Stewart (1993)
Spreadsheet Modified 8/08

Wasatch Regional Landfill DMW
4 to 1 slopes Feb-09

Within Vegetative Layer (silty sand)

	During Heavy Rainfall	Without Heavy Rainfall
Slope Angle, B, (degrees)	14.03	14.03
Ave. Depth of Solution in Cover Layer (ft.)	0*	0
Topsoil Thickness, (ft.)	0	0
Cover Soil Layer Thickness, (ft.)	2.5	2.5
Topsoil Saturated Unit Weight, (pcf)	0	0
Cover Layer Total Unit Wt., (pcf)	100	100
Cover Layer Saturated Unit Weight., (pcf)	115	115
Solution Unit Wt. (pcf)	62.4	62.4
Interface Friction, phi, (degrees)	30	30
Interface Adhesion (psf)	0	0
Earthquake Coef., Ce, (%g)	0.15	0.15
Gas Pressure (psf)	0	0

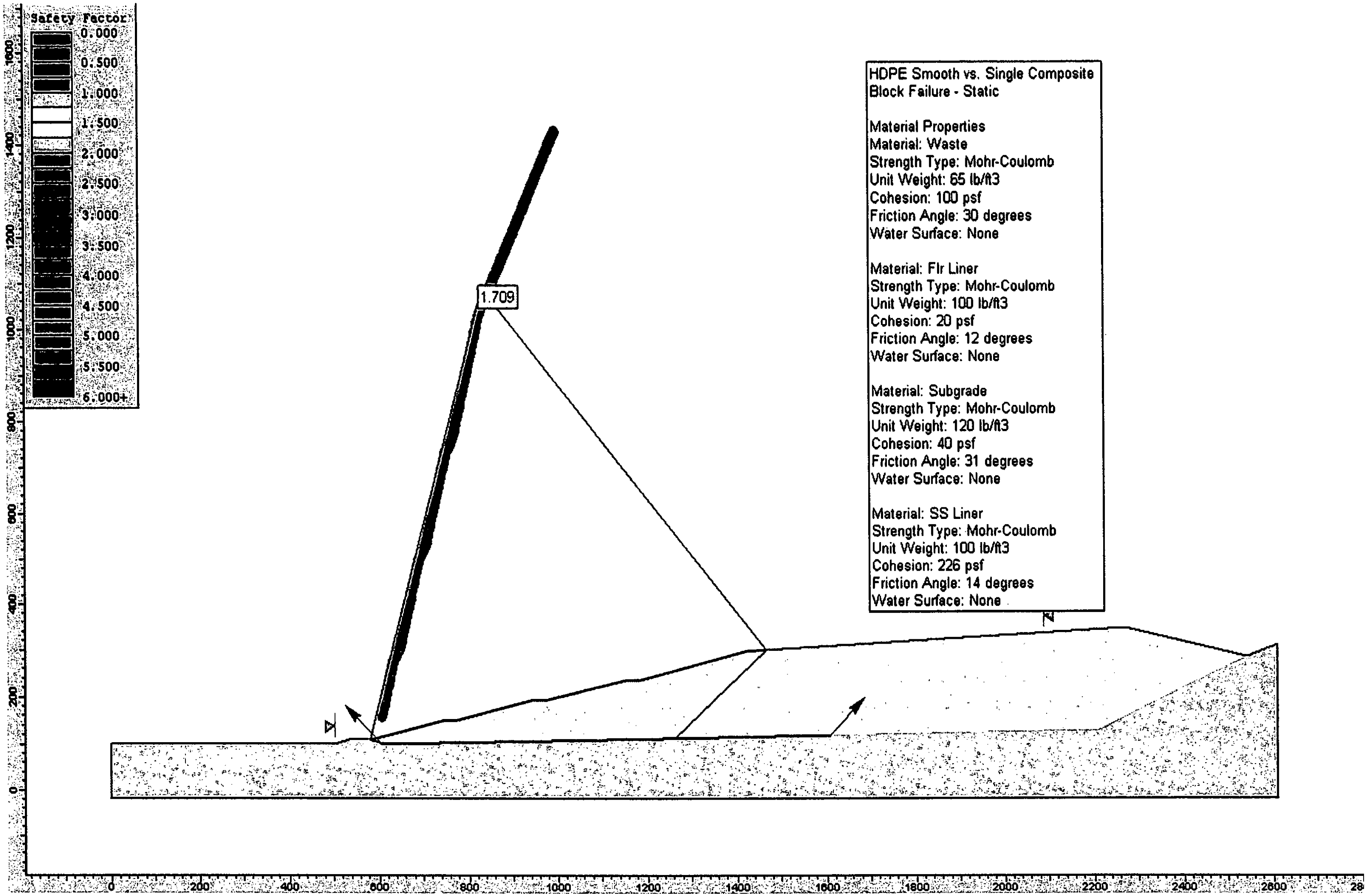
* ET cover is not expected to fully saturate

Sin B	0.2424	0.2424
Cos B	0.9702	0.9702
Tan phi	0.5774	0.5774
Tan B	0.2499	0.2499

STATIC Without Gas Pressure		
Resisting Strength (psf)	140.0	140.0
Driving Stress (psf)	60.6	60.6
Factor of Safety	2.31	2.31

PSEUDO-STATIC Without Gas Pressure		
Resisting Stress (psf)	134.8	134.8
Driving Stress (psf)	97.0	97.0
Factor of Safety	1.39	1.39

Thiel, R.S., and Stewart, M.G., 1993,"Geosynthetic Landfill Cover Design Methodology and Construction Experience in the Pacific Northwest", *Proceedings of Geosynthetics '93, IFAI. Vo. 3,*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2.000
2.500
3.000
3.500
4.000
4.500
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5.500
6.000+
HDPE Smooth vs. Single Composite
Block Failure - Static
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
1.709
1600
1400
1200
1000
800
600
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0
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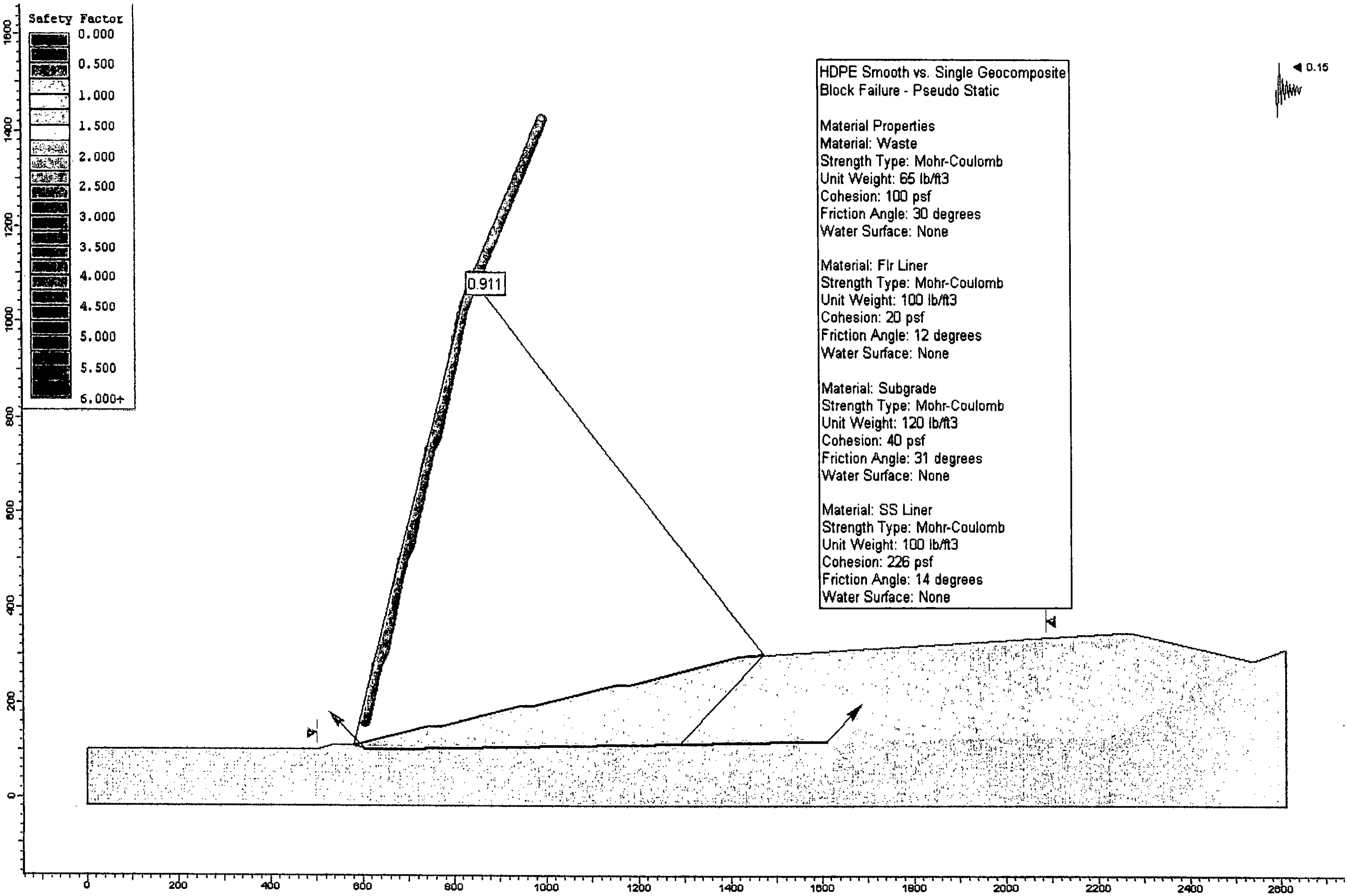
2000
2200
2400
2600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5.500
6.000+
HDPE Smooth vs. Single Geocomposite
Block Failure - Pseudo Static
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
0.911
0.15
1600
1400
1200
1000
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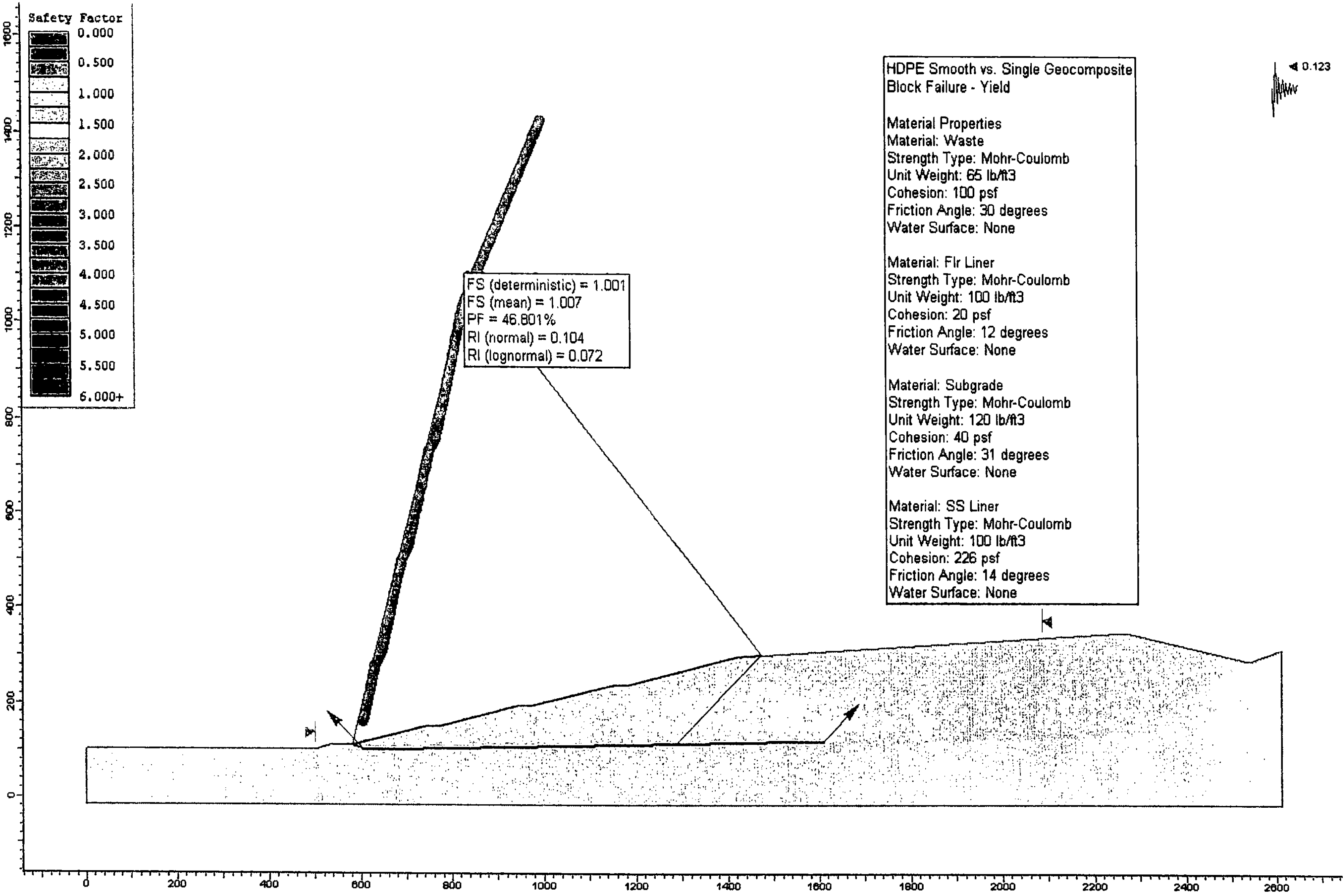

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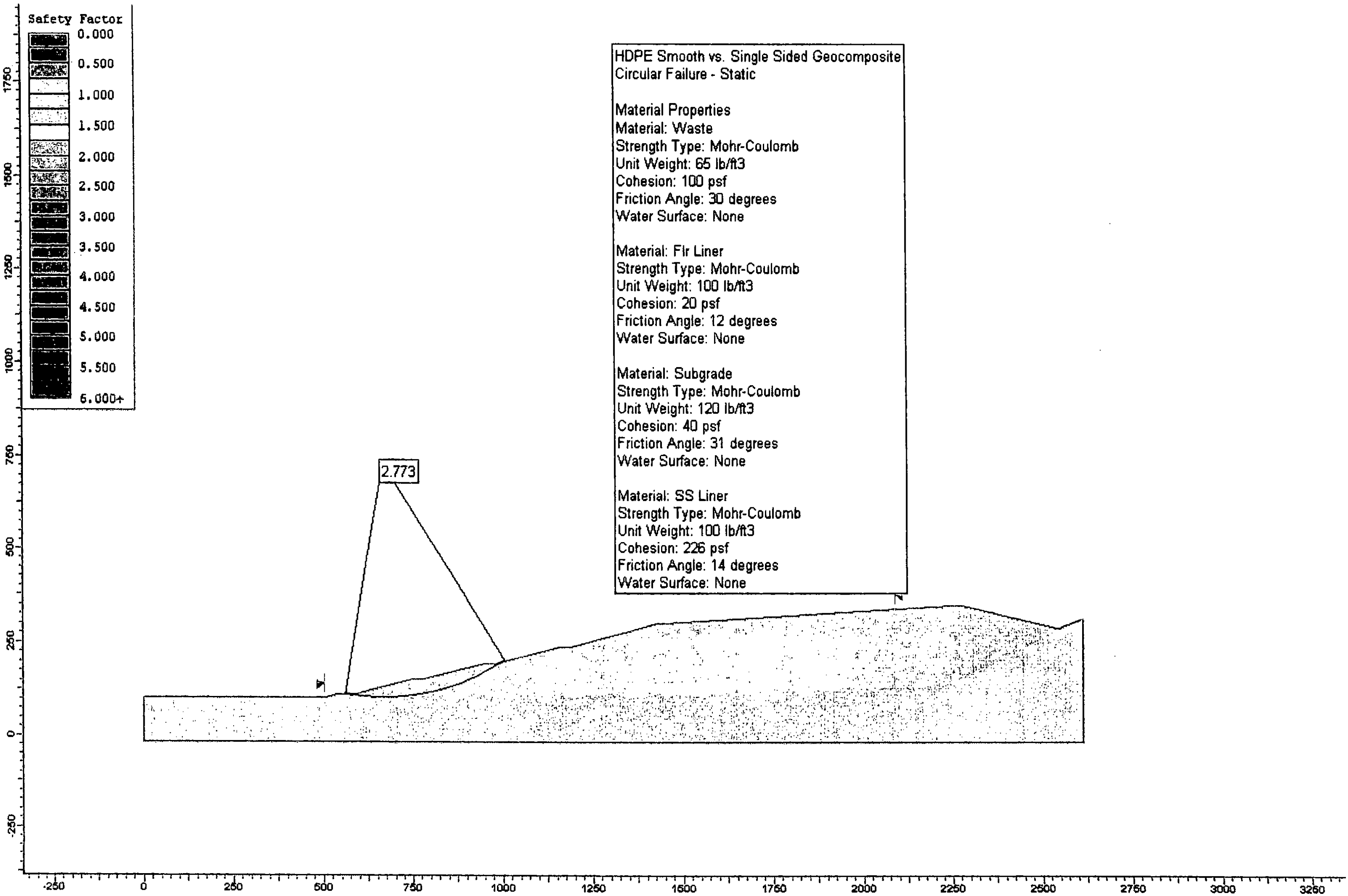
5.000
5.500
6.000+
HDPE Smooth vs. Single Geocomposite
Block Failure - Yield
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
FS (deterministic) = 1.001
FS (mean) = 1.007
PF = 46.801%
RI (normal) = 0.104
RI (lognormal) = 0.072
0.123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5.500
6.000+
HDPE Smooth vs. Single Sided Geocomposite
Circular Failure - Static
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
2.773
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1500
1250
1000
750
500
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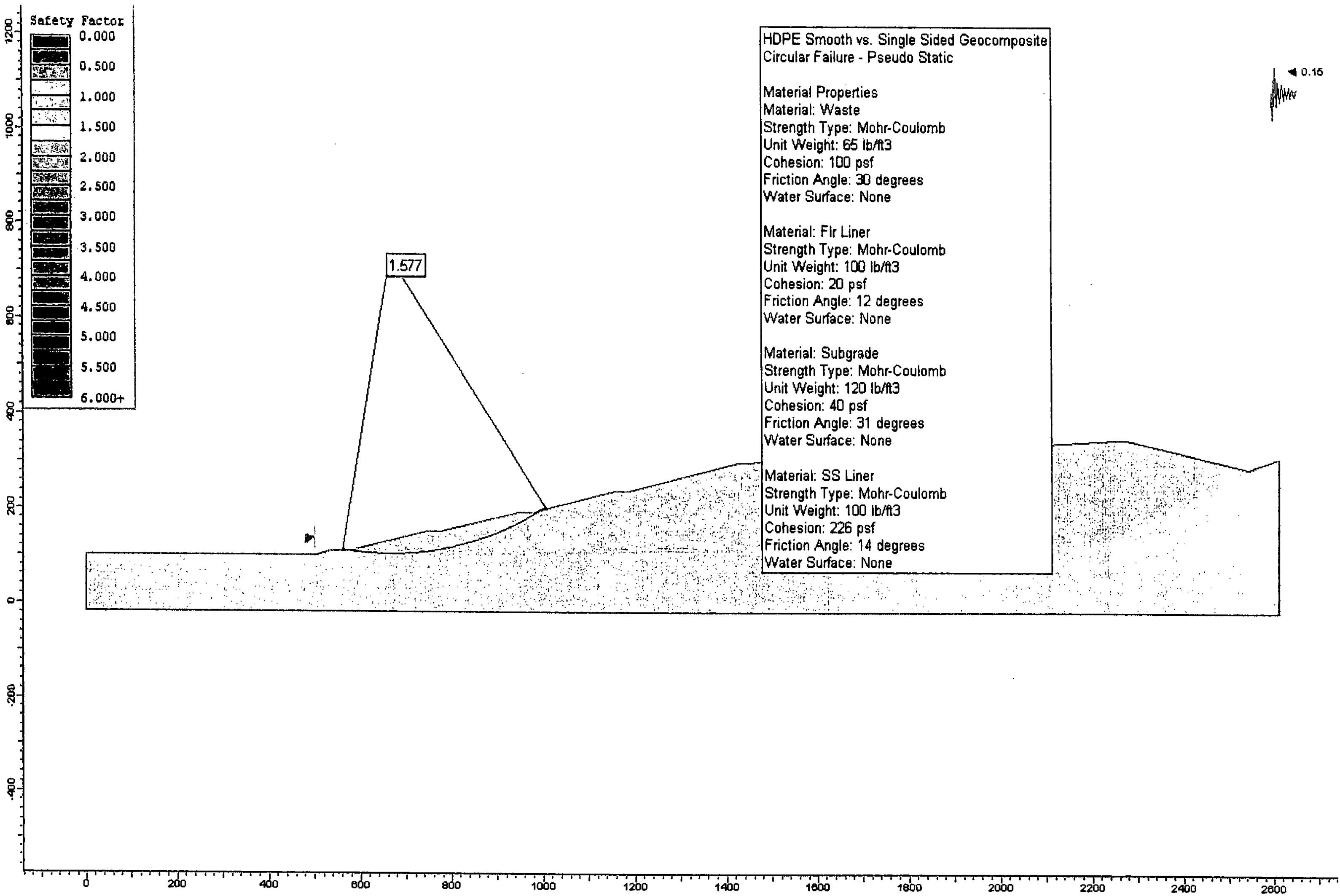

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5.500
6.000+
1.577
HDPE Smooth vs. Single Sided Geocomposite
Circular Failure - Pseudo Static
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
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Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
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1000
800
600
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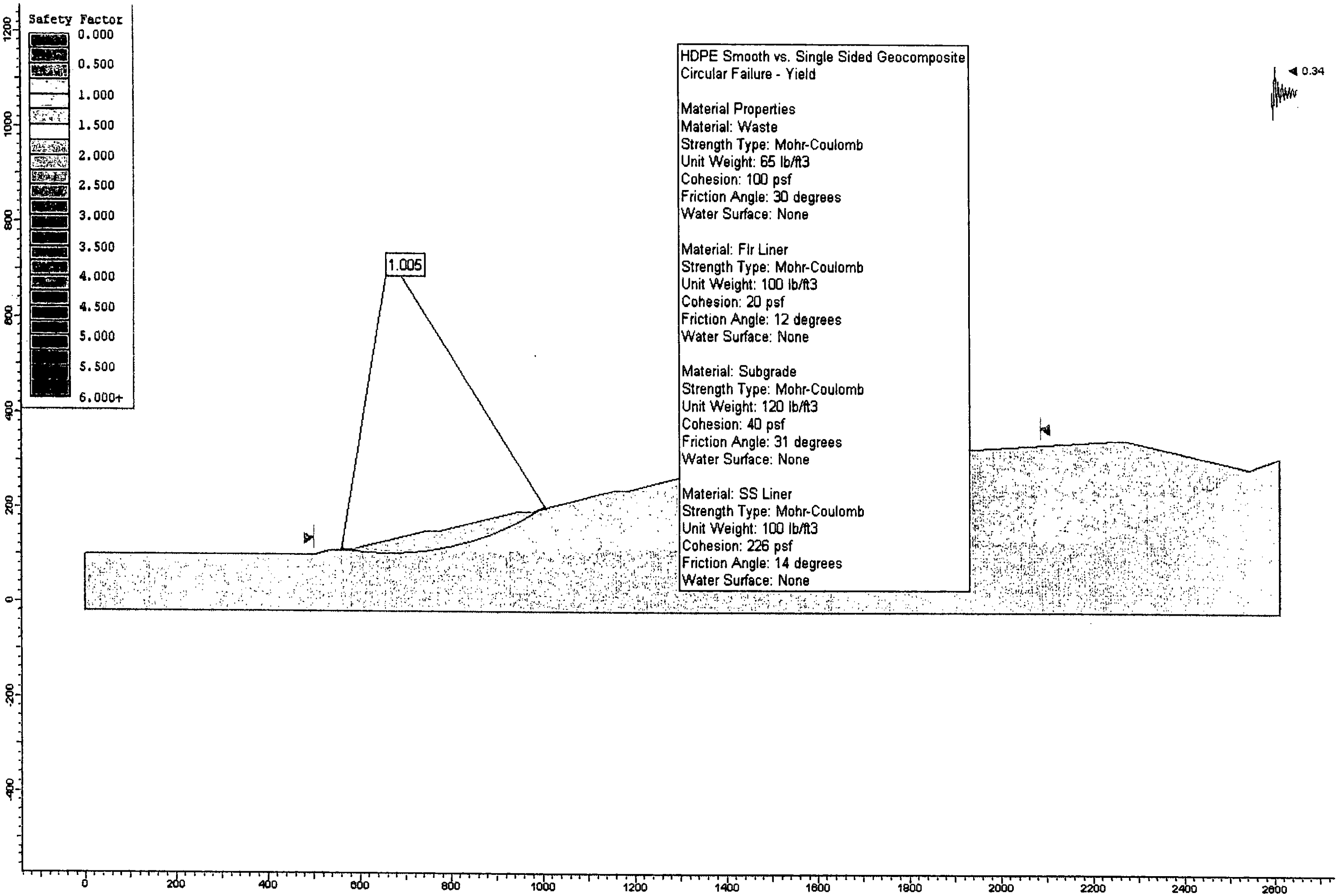

Safety Factor
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1.500
2.000
2.500
3.000
3.500
4.000
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5.000
5.500
6.000+
HDPE Smooth vs. Single Sided Geocomposite
Circular Failure - Yield
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
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Friction Angle: 14 degrees
Water Surface: None
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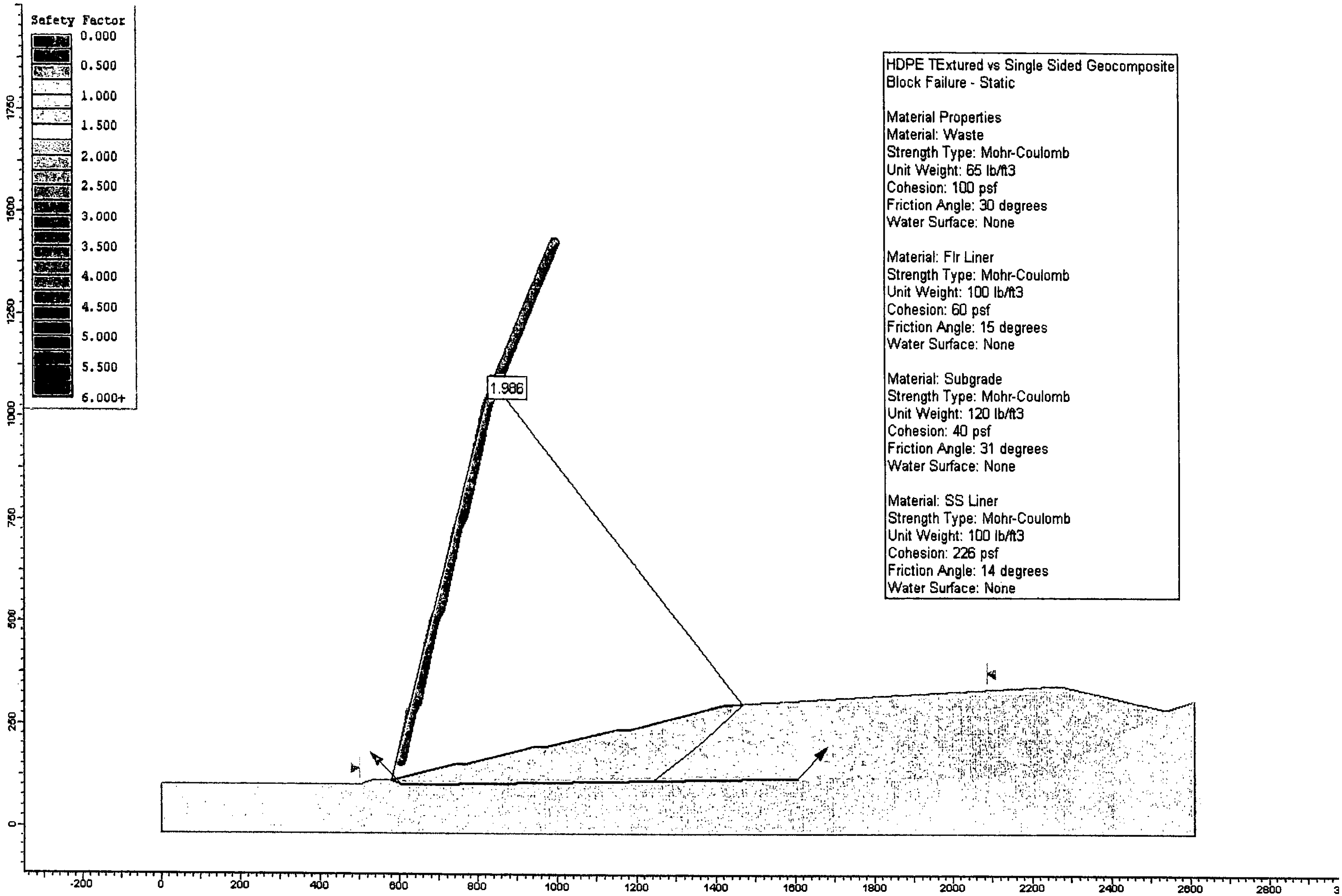
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5.500
6.000+
HDPE TExtured vs Single Sided Geocomposite
Block Failure - Static
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 60 psf
Friction Angle: 15 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
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Water Surface: None
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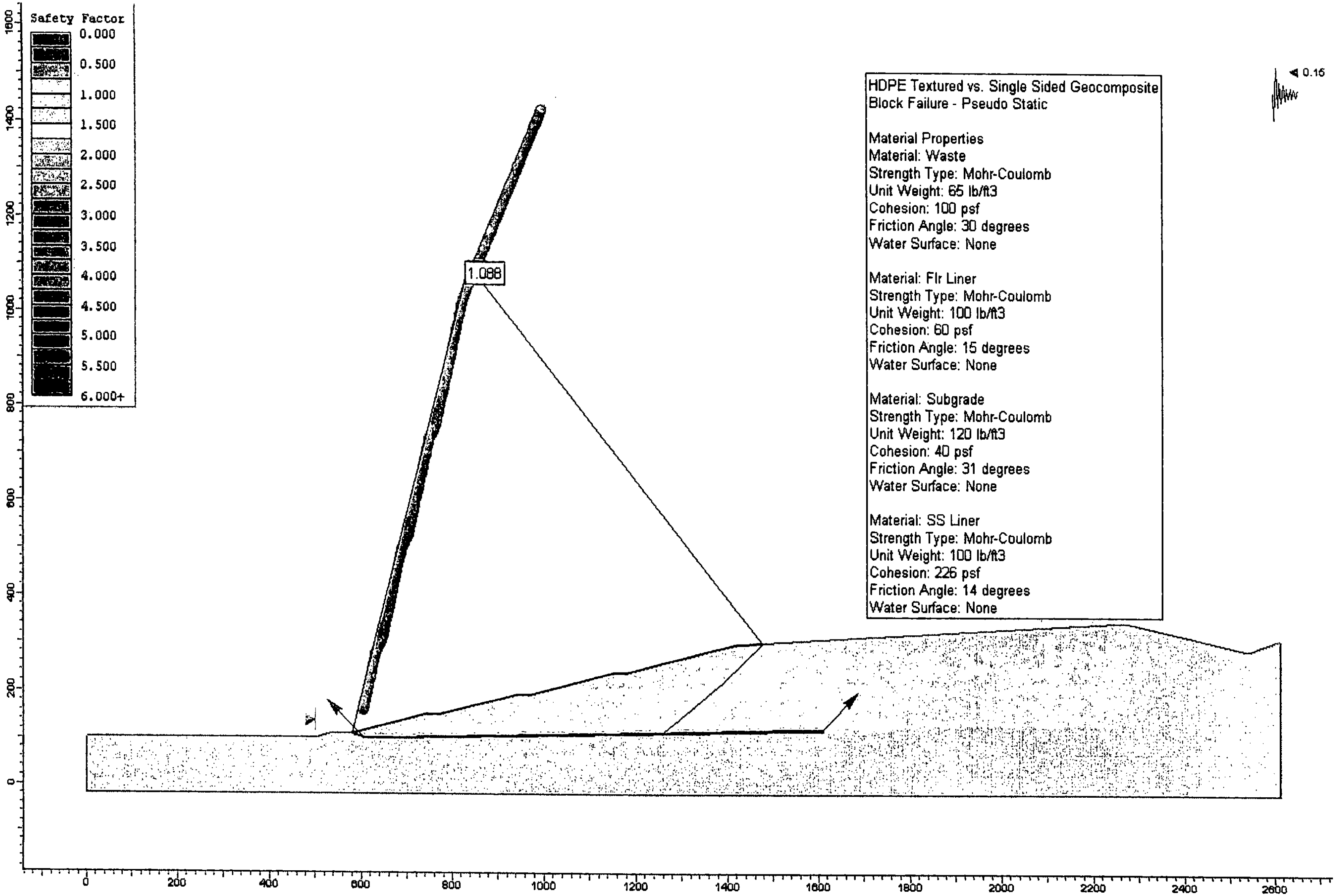
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5.000
5.500
6.000+
HDPE Textured vs. Single Sided Geocomposite
Block Failure - Pseudo Static
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 60 psf
Friction Angle: 15 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
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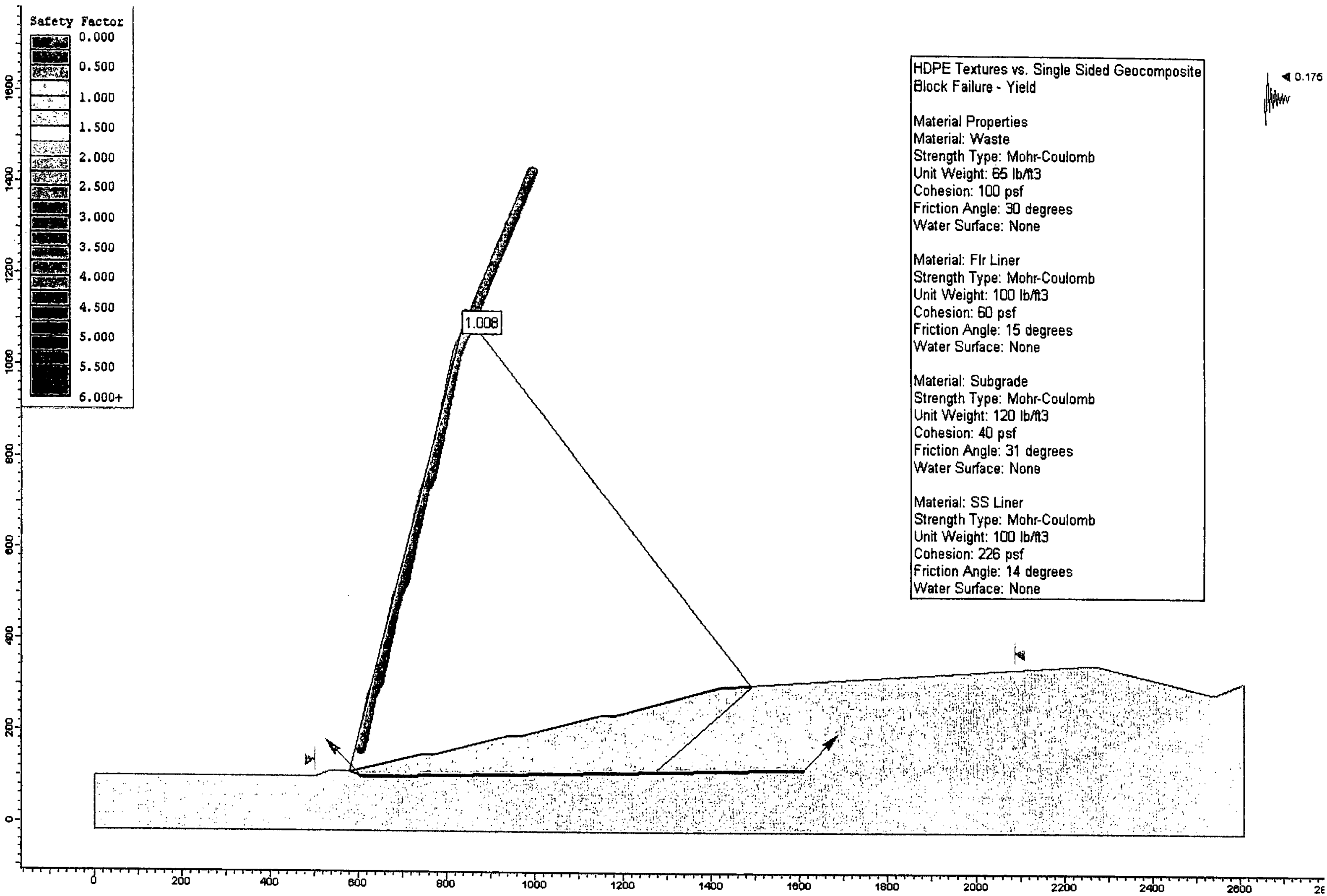
2400
2600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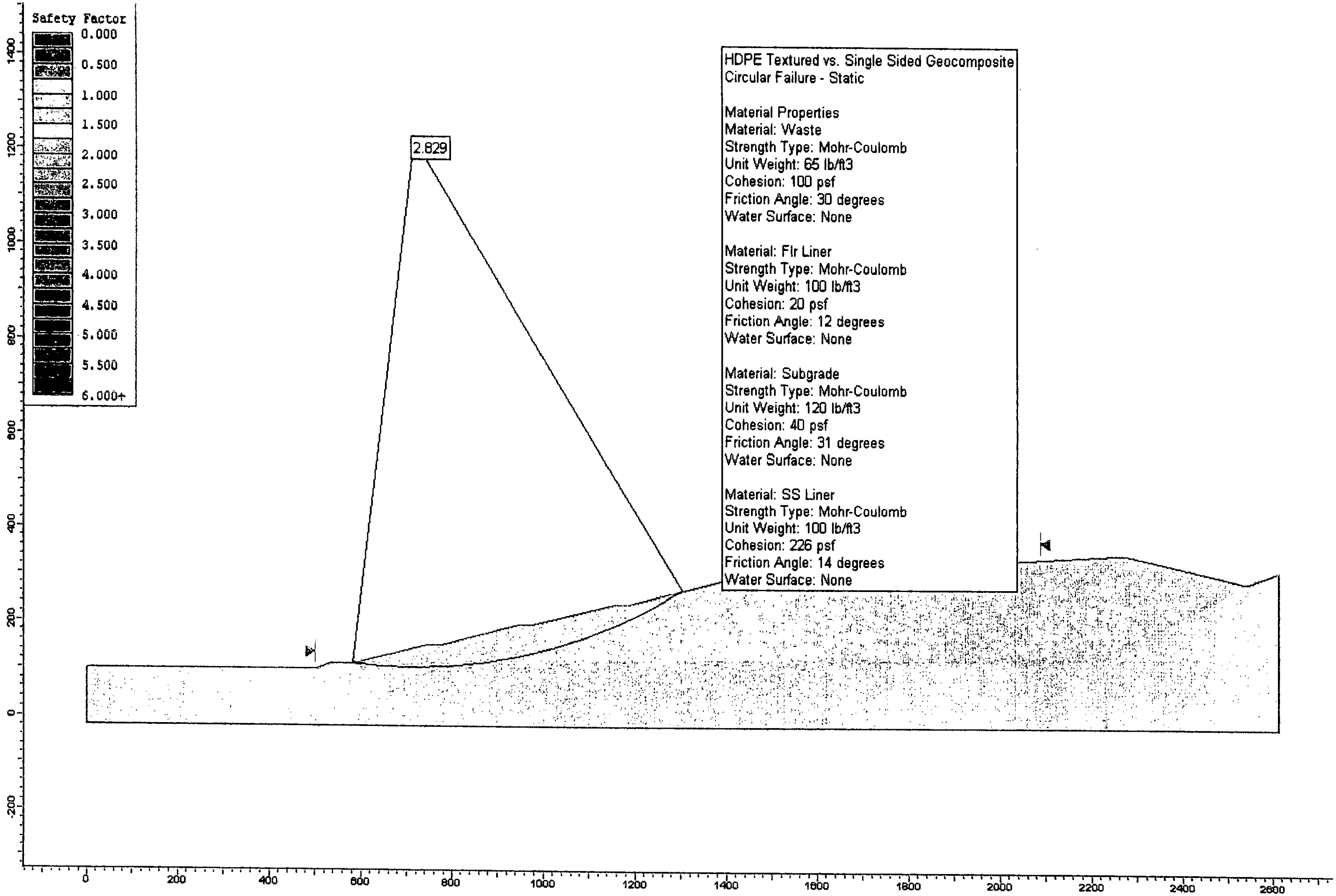
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.008
0.175
HDPE Textures vs. Single Sided Geocomposite
Block Failure - Yield
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 60 psf
Friction Angle: 15 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
1600
1400
1200
1000
800
600
400
200
0
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600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1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
2.829
HDPE Textured vs. Single Sided Geocomposite
Circular Failure - Static
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
1400
1200
1000
800
600
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200
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600
800
1000
1200
1400
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1800
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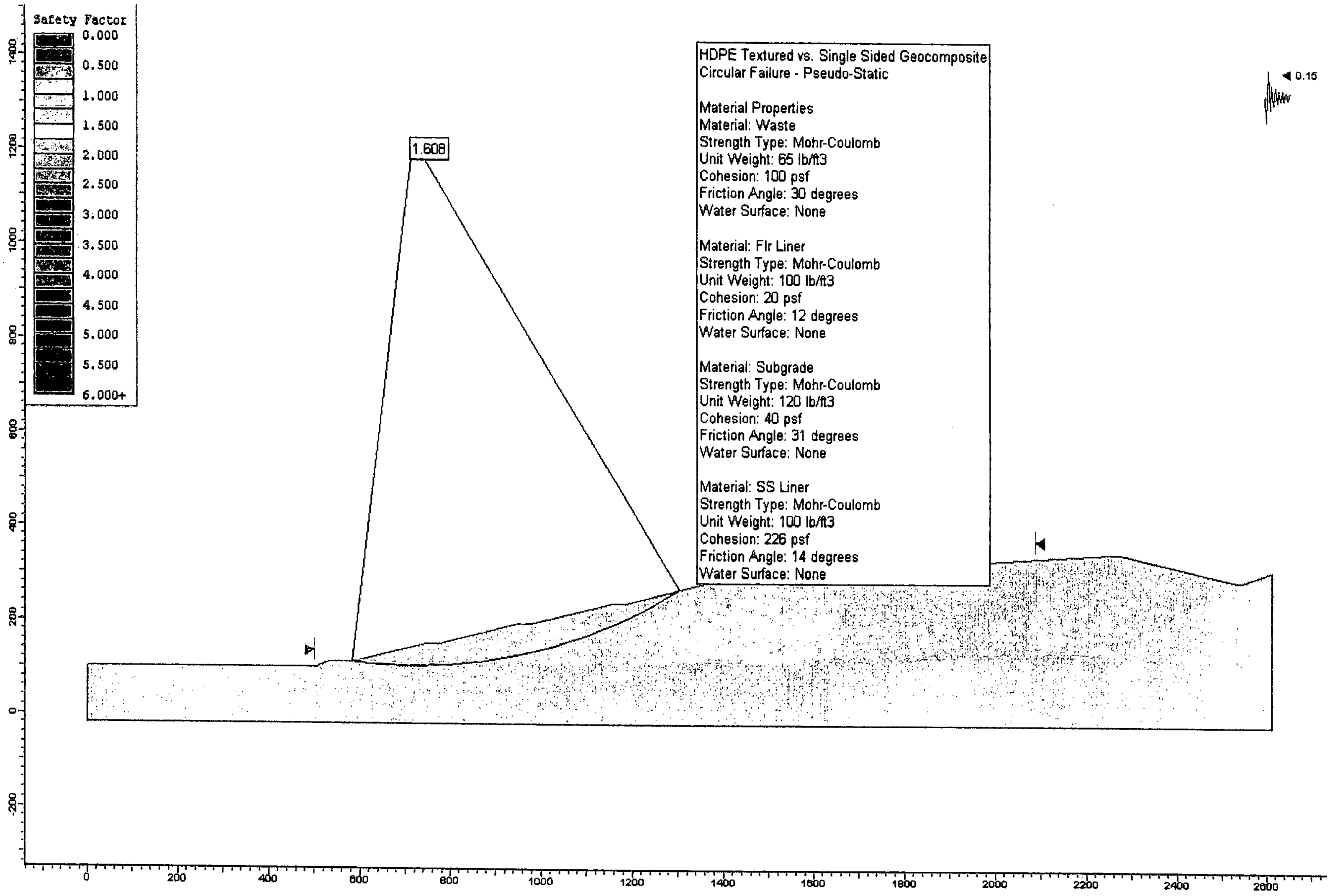
2200
2400
2600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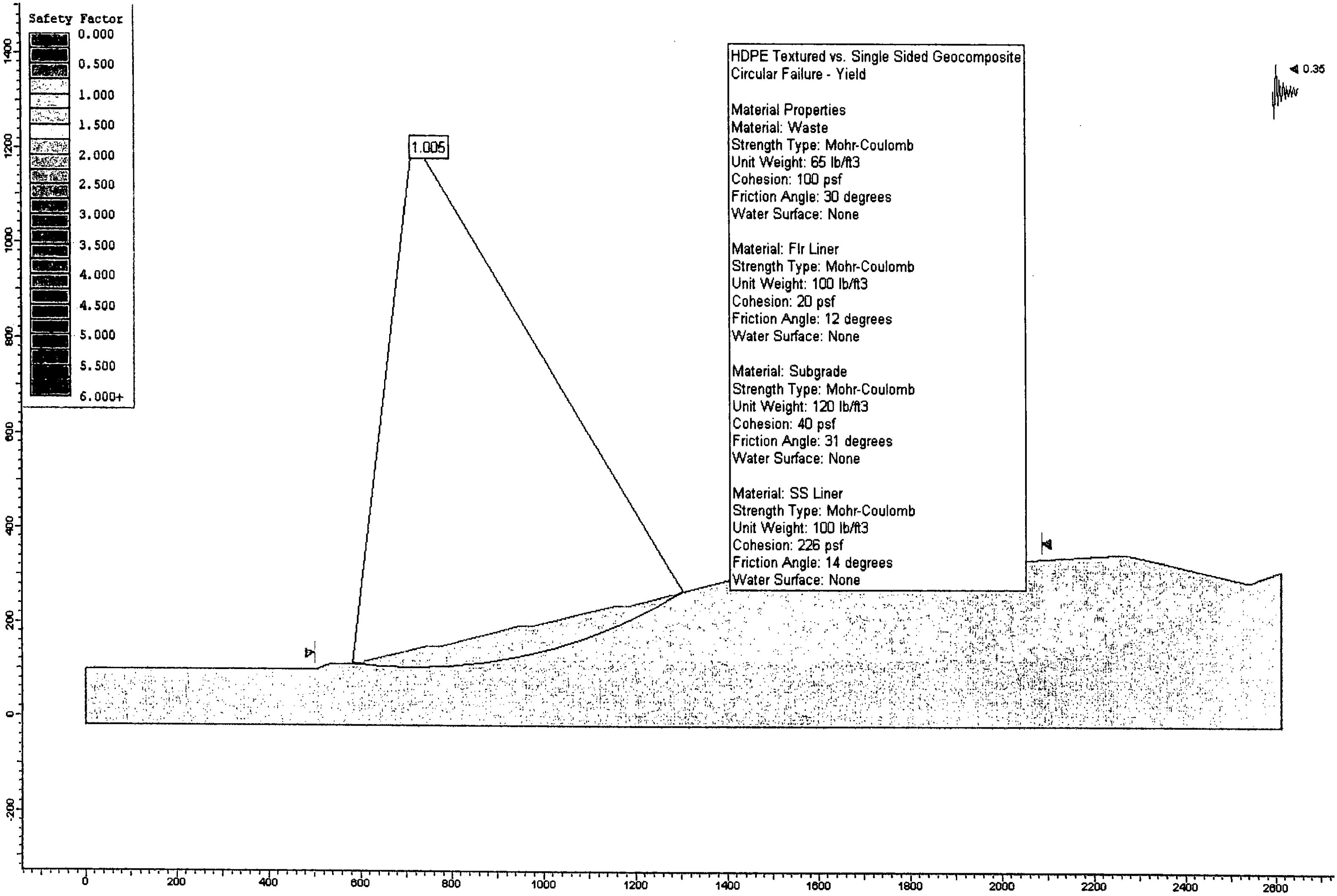
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.608
HDPE Textured vs. Single Sided Geocomposite
Circular Failure - Pseudo-Static
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
0.15
1400
1200
1000
800
600
400
200
0
-200
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
1.005
0.35
HDPE Textured vs. Single Sided Geocomposite
Circular Failure - Yield
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
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Unit Weight: 120 lb/ft3
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Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
1400
1200
1000
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600
400
200
0
-200
0
200
400
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1000
1200
1400
1600
1800
2000
2200
2400
2600

ATTACHMENT 2
LEACHATE COLLECTION AND REMOVAL SYSTEM CALCULATIONS

Transmissivity Calculations

landfilldesign.com

Design of Lateral Drainage System in Landfill - Design Calculator

Problem Statement

The ultimate transmissivity of a geocomposite drainage layer is calculated by two methods:

The first method is based on the McEnroes equations. From the McEnroes equations, the required permeability of a drainage media is calculated. Iteration procedure is used to find the required permeability such that the liquid thickness is equal to the thickness of the liquid collection layer. This permeability multiplied by the thickness of the liquid collection layer result in the required transmissivity. The ultimate geocomposite transmissivity can then be calculated by incorporating the total serviceability factor (product of safety factor and reduction factors).

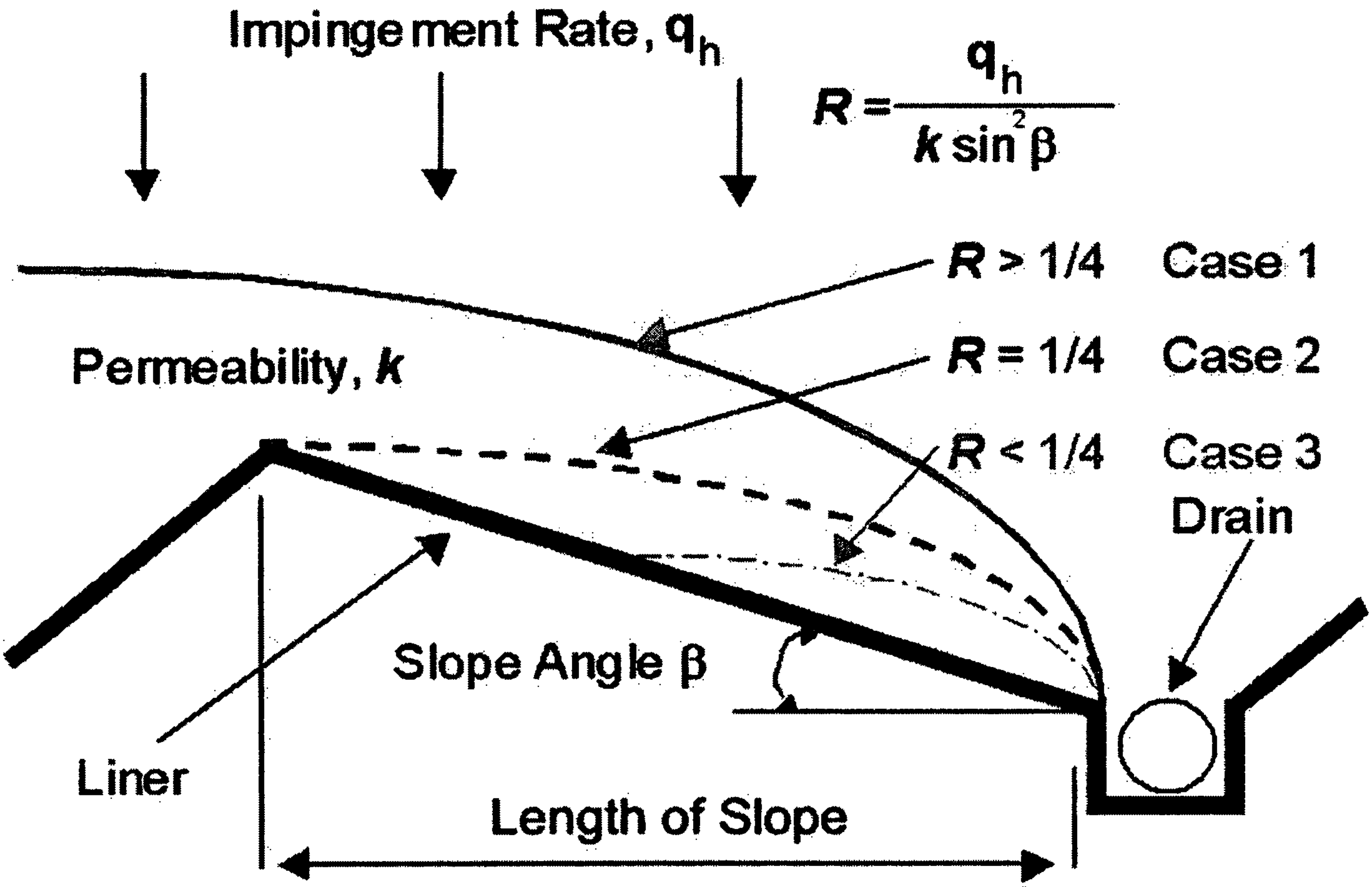
The McEnroe equation requires the input of an impingement rate (q_h), a drainage media permeability (k) and a liner slope (b). This information is used here to find the liquid thickness on the liner.

The McEnroes solutions are for three cases.

1. Case 1 is for a saw-tooth bottom, with the liquid mound overtopping the peak. (R > 1/4)
2. Case 2 has the liquid mound starting at the peak of the saw-tooth. (R = 1/4)
3. Case 3 has the mound starting below the peak of the tooth. (R > 1/4)

McEnroe Equation

$$\frac{t_{LCL}}{L} = \begin{cases} \sin\beta\sqrt{R - RS + R^2S^2}\left[\dfrac{(1-A-2R)(1+A-2RS)}{(1+A-2R)(1-A-2RS)}\right]^{\frac{1}{2A}} \\[2ex] \sin\beta\,\dfrac{R(1-2RS)}{1-2R}\exp\left[\dfrac{2R(S-1)}{(1-2RS)(1-2R)}\right] \\[2ex] \sin\beta\sqrt{R - RS + R^2S^2}\,\exp\left[\dfrac{1}{B}\tan^{-1}\left(\dfrac{2RS-1}{B}\right) - \dfrac{1}{B}\tan^{-1}\left(\dfrac{2R-1}{B}\right)\right] \end{cases}$$



The second method is based on Giroud's equation. The geocomposite's ultimate transmissivity is calculated directly.

Giroud's equation, with great simplicity, produces a very close solution as compared to McEnroe's equations.

Giroud Equation

$$\Theta = \mathrm{TSF}\frac{q_h L}{\sin\beta + \frac{t_{LCL}/L}{TSF}\cos^2\beta}$$

Note: Giroud's equation is based on a factor of safety applied to maximum liquid thickness to ensure unconfined flow.

Required Data

Symbol	Name	Dimensions
S	The liner slope, S = tan b	%
qh	Impingement rate	Length / Time
L	Length of slope measure horizontally	Length
t_{LCL}	Thickness of the Liquid Collection Layer for geocomposite.	Length

FS_d	Overall factor of safety for drainage
RF_{in}	Intrusion Reduction Factor

RF_{cr}	Creep Reduction Factor
RF_{cc}	Chemical Clogging Reduction Factor
RF_{bc}	Biological Clogging Reduction Factor

Input Values

Note: If you do not wish to perform calculations for 3 cases, please leave default data as is.

	Case 1	Case 2	Case 3
S	2.68 %	1 %	1 %
qh	7.11e-6 cm/s	1 cm/s	1 cm/s
L	42.672 m	1 m	1 m
t_{LCL}	60.96 cm	1 cm	1 cm

Factor	Case 1	Case 2	Case 3		Leachate Collection and Removal	Leachate Detection Systems
RF_{in}	1.2	1	1	[1]	1.0 - 1.2	1.0 - 1.2
RF_{cr}	3.5	1	1	[2] Calculate RF_{CR}		
RF_{cc}	1.5	1	1	[3]	1.5 - 2.0	1.1 - 1.5
RF_{bc}	1.3	1	1	[3]	1.1 - 1.3	1.1 - 1.3
FS	2	1	1	[4]	2.0 - 10.0	2.0 - 10.0

Note: The reduction factor values given correspond to the case where the seating time exceeds 100 hours and the boundary conditions due to adjacent materials are simulated in the hydraulic transmissivity test.

Calculate Transmissivity

[1] Intrusion reduction factor from 100 hour to design life. Giroud et. al (2000)
[2] Creep reduction factor from 100 hour to design life (for instance, 30 years). RF_{CR} is determined from 10,000 hour compressive creep test, extrapolated to design life, GRI-GC8 (2001). RF_{CR} is product and normal load specific.
[3] GRI-GC8
[4] FS value = 2-3. Giroud, et. al (2000)
FS value > 10 for filtration and drainage. Koerner (2001)
[5] Note: The calculated transmissivity is corresponding to the case where the seating time is 100 hours and the boundary conditions due to adjacent materials are simulated in the hydraulic transmissivity test.

Solution

Symbol	Name	Dimensions
R	$= q_h/(k \sin^2 b)$	-
Gradient	Gradient	-
θ	Transmissivity = k t_{LCL} TSF	Length^2 / Time

Case 1

McEnroe	Giroud
R = 9.67E-001 **R > 1/4 Case 3**	θ = 1.80E-003 m^2/s
Gradient = 0.03	
θ = 1.02E-003 m^2/s	

Case 2

McEnroe	Giroud
R = 2.35E+000 R > 1/4 Case 3	
Gradient = 0.01	θ = 5.00E-001 m^2/s
θ = 4.26E-001 m^2/s	

Case 3

McEnroe	Giroud
R = 2.35E+000 R > 1/4 Case 3	
Gradient = 0.01	θ = 5.00E-001 m^2/s
θ = 4.26E-001 m^2/s	

Additional Assistance

If you would like to have Advanced Geotech Systems provide material specifications that meet your performance criteria, please fill in the following fields and click the submit button. All information is kept strictly confidential.

Name *

Company

Email Address *

Phone

Project Reference

Comments

*required fields

Submit Design Results

References

"GRI-GC8, Determination of the Allowable Flow Rate of a Drainage Geocomposite". Geosynthetics Research Institute, 2001.

"Designing with Geosynthetics". **R.M. Koerner**, Prentice Hall Publishing Co., Englewood Cliffs, NJ, 1998.

"Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers". **J. P. Giroud**, **J. G. Zornberg** and **A. Zhao**, *Geosynthetics International*, Vol. 7, Nos 4-5.

"Lateral Drainage Design update - part 2". **G. N. Richardson**, J.P. Giroud and **A. Zhao**, *Geotechnical Fabrics Report*, March, 2002

"Maximum Saturated depth over Landfill Liners". B. McEnroe, Journal of Environmental Engineering (Vol. 19, No. 2, March/April, 1993).

Pipe Deflection

PIPE PARAMETERS - AASHTO M294, Type S

- effective radius (in), R = 3.543
- *outside diameter (in), D= 9.45*
- *thickness (in), t = 1.310*
- *unit area of wall (in^2/in), A = 0.128*
- *unit moment of inertia (in^4/in), I =* 0.007
- *flexural modulus (psi), E_f =* 100,000
- *ring compression modulus (psi), E_{rc} =* 100,000
- flexural stiffness (psi), $K_f = 6E_fI/R^3$ = 89
- ring compression stiffness (psi), $K_{rc} = E_{rc}A/R$ = 3,613
- *distance from inner wall to n.a. (in), c =* 0.13

SOIL PARAMETERS - good granular soil

- *mod of soil reaction at 5' of cover (psi), E'_5 =* 1000
- *modulus of soil reaction (psi), E' =* 3,572
- *Poisson's ratio, u =* 0.30
- constr mod (psi), M*=E*(1-u)/((1+u)(1-2u))= 4808
- lateral stress ratio = K = u/(1-u) = 0.429
- sym lateral stress ratio = B = (1/2)(1+K) = 0.714
- antisym lat stress ratio = C = (1/2)(1-K) = 0.286

SOIL/STRUCTURE PARAMETERS (full slippage)

- ring flexibility ratio, UF =(1+K)M*/K_{rc} = 1.90
- bending flexibility ratio, VF = (1-K)M*/K_f = 30.9

STRESS FUNCTION COEFFICIENTS

- constant term, a_0* = 0.205
- cos(2*theta), a_2** = 0.957
- sin(2*theta), b_2** = 0.935

LOAD PARAMETERS

- *unit weight of soil (lb/ft^3) =* 75
- *height of fill above crown (ft) =* 300.0
- surcharge pressure (psi), P = 156.3

RESPONSE OF PIPE WALL

deg c.c.w. from horiz	radial soil press P_r(psi)	radial defl w(in)	tang defl v(in)	circum wall thrust N(#/in)	wall bend mom(M) (#-lb/in)	ring comp stress (psi)	inner bend stress (psi)	outer bend stress (psi)	total stress inner (psi)	total stress outer (psi)
0	83.0	-0.066	0.000	321	29	-2510	-558	5148	-3067	2638
10	83.3	-0.057	0.027	321	27	-2507	-529	4887	-3036	2381
20	84.3	-0.030	0.050	320	23	-2497	-448	4136	-2945	1638
30	85.9	0.010	0.067	318	17	-2483	-323	2984	-2806	501
40	87.7	0.060	0.076	316	9	-2466	-170	1571	-2636	-895
50	89.8	0.114	0.076	313	0	-2447	-7	68	-2455	-2379
60	91.6	0.164	0.067	311	-8	-2430	146	-1345	-2284	-3775
70	93.2	0.204	0.050	309	-14	-2416	270	-2496	-2145	-4912
80	94.2	0.231	0.027	308	-18	-2407	352	-3248	-2055	-5655
90	94.5	0.240	0.000	308	-20	-2403	380	-3509	-2023	-5912
100	94.2	0.231	-0.027	308	-18	-2407	352	-3248	-2055	-5655
110	93.2	0.204	-0.050	309	-14	-2416	270	-2496	-2145	-4912
120	91.6	0.164	-0.067	311	-8	-2430	146	-1345	-2284	-3775
130	89.8	0.114	-0.076	313	0	-2447	-7	68	-2455	-2379
140	87.7	0.060	-0.076	316	9	-2466	-170	1571	-2636	-895
150	85.9	0.010	-0.067	318	17	-2483	-323	2984	-2806	501
160	84.3	-0.030	-0.050	320	23	-2497	-448	4136	-2945	1638
170	83.3	-0.057	-0.027	321	27	-2507	-529	4887	-3036	2381
180	83.0	-0.066	0.000	321	29	-2510	-558	5148	-3067	2638

CALCULATION OF RING SHORTENING

deg c.c.w. from horiz	ring comp stress (psi)	ring comp strain (in/in)	ring shortening (in)
0	-2510	-0.02509895	-0.0155
10	-2507	-0.0251	-0.0155
20	-2497	-0.02497426	-0.0154
30	-2483	-0.02483247	-0.0154
40	-2466	-0.02465853	-0.0152
50	-2447	-0.02447343	-0.0151
60	-2430	-0.02429949	-0.0150
70	-2416	-0.0241577	-0.0149
80	-2407	-0.02406515	-0.0149
90	-2403	-0.024033	-0.0149
100	-2407	-0.02406515	-0.0149
110	-2416	-0.0241577	-0.0149
120	-2430	-0.02429949	-0.0150
130	-2447	-0.02447343	-0.0151
140	-2466	-0.02465853	-0.0152
150	-2483	-0.02483247	-0.0154
160	-2497	-0.02497426	-0.0154
170	-2507	-0.0251	-0.0155
180	-2510	-0.02509895	-0.0155
		SUM (l/2 circle) =	-0.2890

COMMENTS

1. This is 8" diameter ADS Type C

2. Flexural and compressive modulus are taken as 100,000 psi (HDPE typical).

3. Typical E'_5 values (in psi) for various soils are listed in the table below:

Type of soil	Standard AASHTO Relative Compaction 85%	90%	95%
Fine-grained soils with less than 25% sand (CL, ML, DL-ML)	500	700	1000
Coarse-grained soils with fines (SM, SC)	600	1000	1200
Coarse-grained soils with little or no fines (SP, SW, GP, GW)	700	1000	1600

MISC CALCS

- Vertical deflection (%) = 6.78
- Horizontal deflection (%) = -3.74
- Critical Buckling Pressure (psi), P_{cr}= 226.4
- Radial Soil Pressure at Crown (psi), P_{act}= 94.5
- Arc length of each sector (in) = 0.6184

CIRCUMFERENCE SHORTENS= -0.58 inches

Calculations by: DMW
Geotechnical Engineer

ATTACHMENT 3
ALTERNATIVE FILL PLAN STABILITY EVALUATION

ALTERNATIVE FILL PLAN STABILITY EVALUATION
of the
WASATCH REGIONAL LANDFILL
Tooele County, Utah

Prepared for:

ALLIED WASTE INDUSTIRES, INC
111 West Highway 123
East Carbon, Utah

Prepared by:

VECTOR
ENGINEERING, INC.

An **Ausenco** group company

143E Spring Hill Drive
Grass Valley, CA 95945
(530) 272-2448

Project No. 061204.11
February 2009

TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

1.1 Purpose

The purpose of this analysis was to evaluate the slope stability for alternative liner systems and final fill configurations without benches for the Wasatch Regional Landfill (WRL). Stability analyses were conducted on several landfill configurations to evaluate the stability of the landfill with benches constructed in the final cover rather than benched into the waste.

1.2 Scope of Work

Vector's scope of work included the evaluation of the final liner system options and alternative waste fill configurations for the WRL. Slope stability analyses were performed to ensure the static and pseudo-static stability of the system, and included the following critical design elements:

1. A maximum overall waste slope of 4 horizontal to 1 vertical (4H:1V) *without* benches, with a top deck slope of approximately 5%.
2. Side slopes lined with textured geomembrane and high-strength geosynthetic clay liner (GCL).
3. A floor-liner system comprised of GCL, either smooth or textured geomembrane, and a geocomposite.

The work tasks performed for this study included the following:

1. *Slope Stability Analyses.* Limit-equilibrium slope stability analyses were performed for an idealized cross section of the landfill with no benches in the waste. Slope stability was evaluated for static and pseudo-static (earthquake) conditions.
2. *Displacement Analyses.* Based on the results of the pseudo-static stability analyses, potential displacements were estimated for the design earthquake magnitude.
3. *Report Preparation.* This report summarizes the results and conclusions for each of the tasks listed above.

1.3 Location and General Description

The WRL is located at 8833 North Rowley Road, North Skull Valley, Utah; west of the Great Salt Lake and adjacent to the east side of the Lakeside Mountain Range in Tooele County. The WRL will consist of eleven phases covering approximately 793 acres and will have an ultimate capacity of approximately 160 million cubic yards.

In the final configuration, the waste slopes will be graded at a maximum slope of 4H:1V, with a top deck slope of approximately 5 percent. This evaluation investigates the stability at shallower slopes (i.e. 4.5H:1V and 5.65H:1V) and without benches in the waste material. The highest slope is located on the east side of the landfill running in a north-south direction, having a vertical slope height of approximately 200 ft.

The side-slope liner system and floor liner system configurations used in this stability evaluation are discussed in the *Waste Fill Stability Evaluation of the Wasatch Regional Landfill, Tooele County, Utah* (Vector, 2009) report. Our evaluation considers two floor liner systems configurations, one with a smooth HDPE geomembrane, like the system currently installed at WRL, and one configuration utilizing textured HDPE geomembrane for improved stability.

2.0 SUBSURFACE INVESTIGATION AND CONDITIONS

2.1 Field Investigation

Previous geotechnical investigations for the WRL were conducted by AGEC (2004, 2005) and Kleinfelder (2004). In addition, Vector conducted logging and sampling of four soils from test pits excavated in 2006. Classification tests were performed for the samples, including initial moisture (ASTM D-2216), particle size analysis (ASTM D-422), and Atterberg limits (ASTM D-4318).

2.2 Laboratory Testing

For the purpose of this study, additional laboratory testing was not required. Material shear strength properties were determined from the laboratory testing performed by Vector in April 2008. LSDS tests were completed to obtain shear strength properties for the critical interfaces. Laboratory test results are located in Appendix A of the Vector report *Waste Fill Stability Evaluation of the Wasatch Regional Landfill, Tooele County, Utah* (Vector, 2009).

2.3 Subsurface Conditions

Subsurface information presented within this report was obtained from the Geotechnical Investigation Permit Modification prepared by AGEC (2004) for the WRL. Subsurface conditions at the site were characterized by exploratory borings drilled by AGEC and the subsurface information reported by Kleinfelder and Vector. The subsurface profile generally consists of clay, silt and fine sand on the lower elevation portions of the site, with coarser grained materials present at higher elevations. Limestone bedrock was encountered in boring B-1 (AGEC, Dec. 2004) at a depth of 143 ft.

3.0 FAULTING, SEISMOLOGY & EARTHQUAKE GROUND MOTION

A complete seismic hazard evaluation for WRL was conducted as part of Vector's stability report *Waste Stability Evaluation of the Wasatch Regional Landfill, Tooele County, Utah* (Vector, 2009). Deterministic seismic hazard analyses were conducted for 12 fault sources within a 160 km radius of the WRL to provide the potential ground motion seismic evaluation of the waste fill stability.

3.1 Design Basis Earthquake Event

As determined from the seismic hazard evaluation, the site historically experienced an estimated acceleration of 0.10 g during the event of March 12, 1934, which was the most critical for the site. Based on the risks associated with the Stansbury Fault, a site acceleration of 0.436 g is considered possible. From the probabilistic evaluation, a peak horizontal ground acceleration of 0.435 g was estimated for a 2% probability of exceedance in a 50 year exposure period.

Seed (1979) suggested that to ensure that displacements will be acceptably small, it is only necessary to perform a pseudo-static screening analysis for a seismic coefficient of 0.1 g for earthquakes up to a magnitude 6.5 or 0.15 g for earthquakes up to a magnitude 8.5, and obtain a factor of safety of 1.15 or greater. This procedure is only acceptable for site soils that are not vulnerable to excessive strength loss or pore pressure development. Both field and laboratory experience indicate that clayey soils, dry sands and in some cases dense saturated sands will not lose substantial resistance to deformation as a result of earthquake loading (Seed, 1979).

Based on Vector's seismic hazard analyses (Vector, 2009) and on Seed's (1979) procedure, the design earthquake we have chosen for this site would be from a magnitude 6.9 event on the Stansbury fault. Therefore, a site horizontal seismic coefficient, k_h, of 0.15g was chosen, based on Seed (1979), to be used as a pseudo-static screening value.

4.0 STABILITY ANALYSIS

4.1 General

Vector conducted stability analyses for the WRL for both static and pseudo-static conditions. Pseudo-static analyses were performed to determine the pseudo-static screening factor of safety and the yield acceleration for the slope condition analyzed. Failure surfaces through the waste and along the geomembrane liner were evaluated to determine the factor of safety for slope stability. The cross-section analyzed is located in the northern portion of the WRL and represents the most critical slope of the landfill. The analyzed cross section is presented in Appendix A.

The computer program SLIDE 5, developed by Rocscience, Inc (2003), was used for the analyses to determine the factors of safety and probabilities of failure. Spencer's Method of slices was used in the analysis to obtain the factor of safety. The factor of safety can be defined generally as the resisting forces divided by the driving forces. A factor of safety of 1.0 or less indicates that the slope is potentially unstable. Several search routines were used to evaluate tens of thousands of potential failure surfaces for each case analyzed.

Both static and pseudo-static analyses were performed for circular and non-circular surfaces. The pseudo-static analyses subject the two-dimensional sliding mass to a horizontal acceleration equal to a horizontal earthquake coefficient, k_h, multiplied by the acceleration of gravity. As described in section 4.1, a k_h of 0.15 was used as in our pseudo-static analyses and required a pseudo-static factor of safety of 1.15.

4.2 Material Properties

The material properties of the various components of the landfill needed to perform static and pseudo-static slope stability analyses (e.g. unit weight and shear strength parameters) were obtained from Vector's stability report *Waste Fill Stability*

Evaluation of the Wasatch Regional Landfill, Tooele County, Utah (Vector, 2009). Table 1 shows a summary of the average material properties used for the analyses.

TABLE 1
SUMMARY OF AVERAGE MATERIAL PROPERTIES
USED IN STABILITY ANALYSES

SLOPE LINER SYSTEM	ANALYZED CRITICAL INTERFACE	TOTAL UNIT WEIGHT (PCF)	COHESION (PSF)	INTERNAL ANGLE OF FRICTION (DEGREES)
	Compacted Fill (Subgrade)	120	40	31
	Municipal Solid Waste (MSW)	65	100	30
Side Slope Liner GCL vs. Double Textured HDPE Geomembrane	Textured HDPE Geomembrane/ GCL	100	226[A]	14[A]
Floor Liner - Option 1 GCL vs. Double Smooth HDPE Geomembrane vs. Single Sided Geocomposite	Smooth HDPE Geomembrane/ Single Sided Geocomposite	100	20[A]	12[A]
Floor Liner – Option 2 GCL vs. Double Textured HDPE Geomembrane vs. Single Sided Geocomposite	Textured HDPE Geomembrane / Single Sided Geocomposite	100	60[A]	15[A]

A – From statistical analysis based on typical laboratory test results from similar liner interfaces.

4.3 Results of the Stability Analyses

Circular and non-circular surfaces along the waste and liner interface, respectively, were evaluated using Spencer's method to calculate the FOS. The results of the stability analyses are summarized in Table 2. The critical failure surfaces originated near the toe of the waste slopes and day-lighted near the crest. The output presents the material properties, and locations of the critical shear surfaces with the lowest factor of safety (see Appendix A). The minimum factor of safety calculated in the pseudo-static analyses for the two liner system options was 0.89. Based on these results, seismic displacement analyses were performed.

The yield acceleration (k_y) of the landfill mass was calculated for both liner system configurations. The yield acceleration is defined as the horizontal acceleration that, when applied to the slope in the limit equilibrium (seismic) analyses, results in a pseudo-static factor of safety equal to one. The yield acceleration was determined using the Spencer method and the results are shown in Table 2. The output files from SLIDE 5 for these analyses are included in Appendix A.

TABLE 2
SUMMARY OF SLOPE STABILITY RESULTS FOR ALTERNATIVE LINER SYSTEMS AND WASTE FILL CONFIGURATIONS – NO BENCHES

FLOOR LINER SYSTEM	SLOPE H:V	FACTOR OF SAFETY (NON-CIRCULAR)		FACTOR OF SAFETY (CIRCULAR)		YIELD ACCEL	DISPLACEMENT	
		STATIC	SEISMIC	STATIC	SEISMIC	(G)	IN.	ACCEPTABLE?
With Smooth Geomembrane	4:1	1.58	0.89	2.58	1.56	0.11	0.2	Yes
	4.5:1	1.70	0.91	2.76	1.70	0.122	0.03	Yes
	5.65:1	1.96	0.96	3.34	1.76	0.137	0.0	Yes
With Textured Geomembrane	4:1	1.82	1.05	2.58	1.56	0.165	0.0	Yes

The yield acceleration was used in displacement analyses to estimate the permanent displacement of the landfill that could occur from the design seismic event. The method chosen for these analyses was the "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills," by Bray et al. (1998). This method uses chart solutions to estimate the displacement for earthquake accelerations which are greater than the yield acceleration.

The design earthquake would have a magnitude of 6.9. Based on the earthquake hazard analyses, the design site acceleration would be from a near field event on the Stansbury Fault zone. This event would result in a peak horizontal ground acceleration (PHGA) of 0.436 g at the site. In theory, the landfill will displace during a seismic event when the site acceleration exceeds the yield acceleration.

The yield acceleration for floor-liner Option 1 (the weaker of the two options) was 0.89 g. The analyses show that base sliding of the landfill during the design earthquake would result in top displacements for both options (1 and 2) would be less than 1 inch. For lined landfills, displacements less than or equal to 12 inches are generally considered acceptable (Kavazanjian 1999).

4.4 Conclusions Regarding Slope Stability

A factor of safety equal to or greater than 1.50 and 1.15 is generally considered acceptable for static conditions and pseudo-static conditions, respectively. Under static conditions the section analyzed showed an acceptable factor of safety for all liner configuration options. However, during an earthquake, displacement is possible since the pseudo-static factor of safety was less than 1.15 in both liner configurations. Therefore, a displacement analysis was performed to determine the potential displacement of the waste mass. The seismic displacement analyses indicate that permanent displacements of the landfill from the design seismic event would be small (less than 1 inch).

5.0 CONCLUSIONS

Vector performed slope stability analyses for the WRL based on the conceptual design of the landfill, preliminary soils data and historical seismicity near the site. Circular and non-circular failure surfaces through the waste and the critical liner interface were evaluated to determine the factor of safety for stability. For static conditions, the results of the stability analyses indicate that the landfill will remain stable for both floor liner configurations (smooth and textured HDPE geomembrane) and for all slope angles considered (4:1, 4.5:1 and 5.65:1) without benches in the waste material. For the pseudo-static conditions, the factor of safety for slope stability drops below 1.15, and therefore, a displacement analysis was performed. The displacement estimated from the seismic analysis for the weaker liner condition (smooth geomembrane) ranged from 0.0 in. to 0.2 in., which is considered acceptable (Kavazanjian 1999).

6.0 LIMITATIONS

The recommendations presented in this report are based upon understanding of the project, a field investigation, and the information provided by WRL. This report was prepared in accordance with generally accepted soils and foundation engineering practices applicable at the time the report was prepared. Vector makes no other warranties, either expressed or implied, as to the professional opinions and conclusions provided.

7.0 REFERENCES

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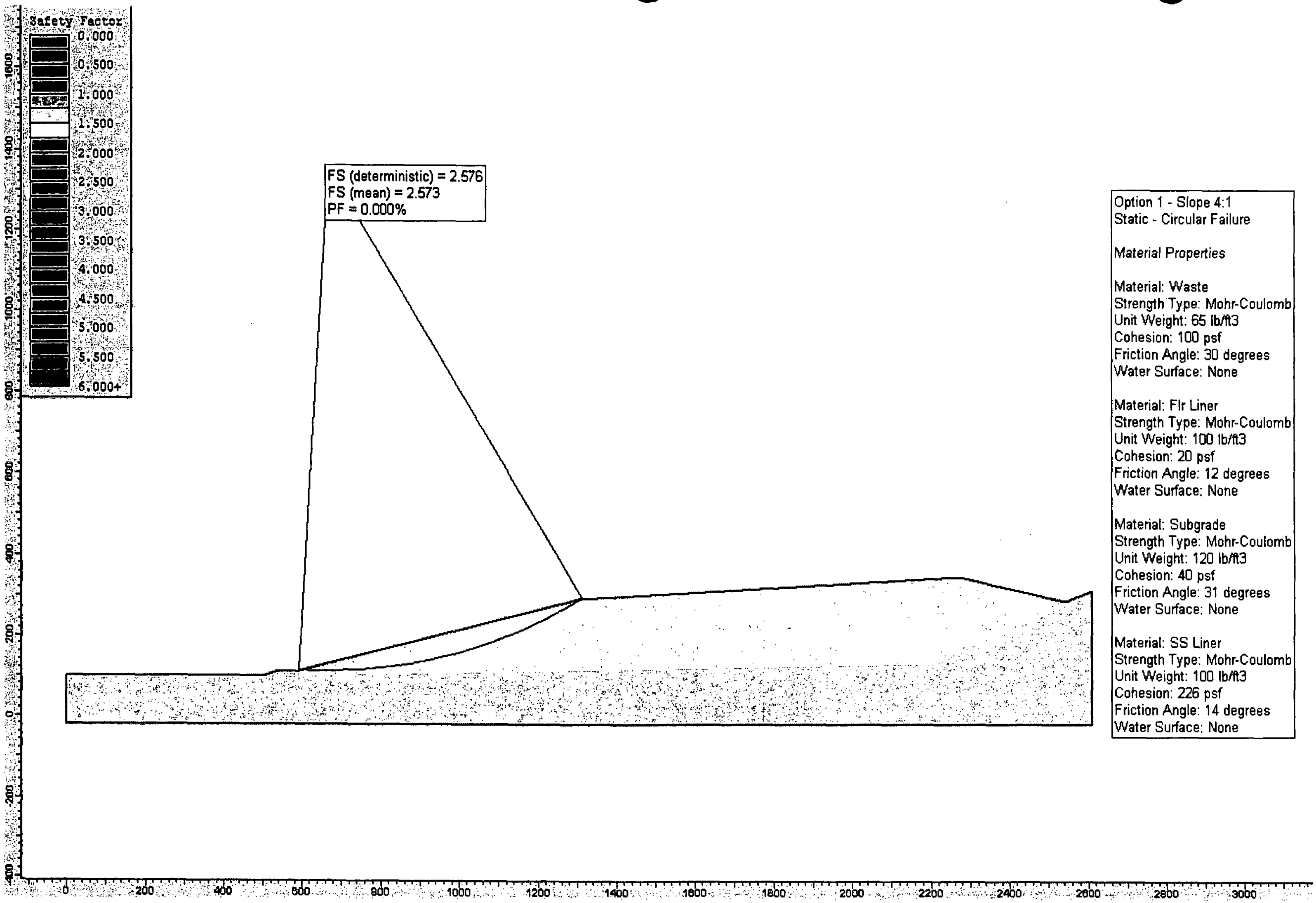
APPENDIX A
SLIDE OUTPUT FILES



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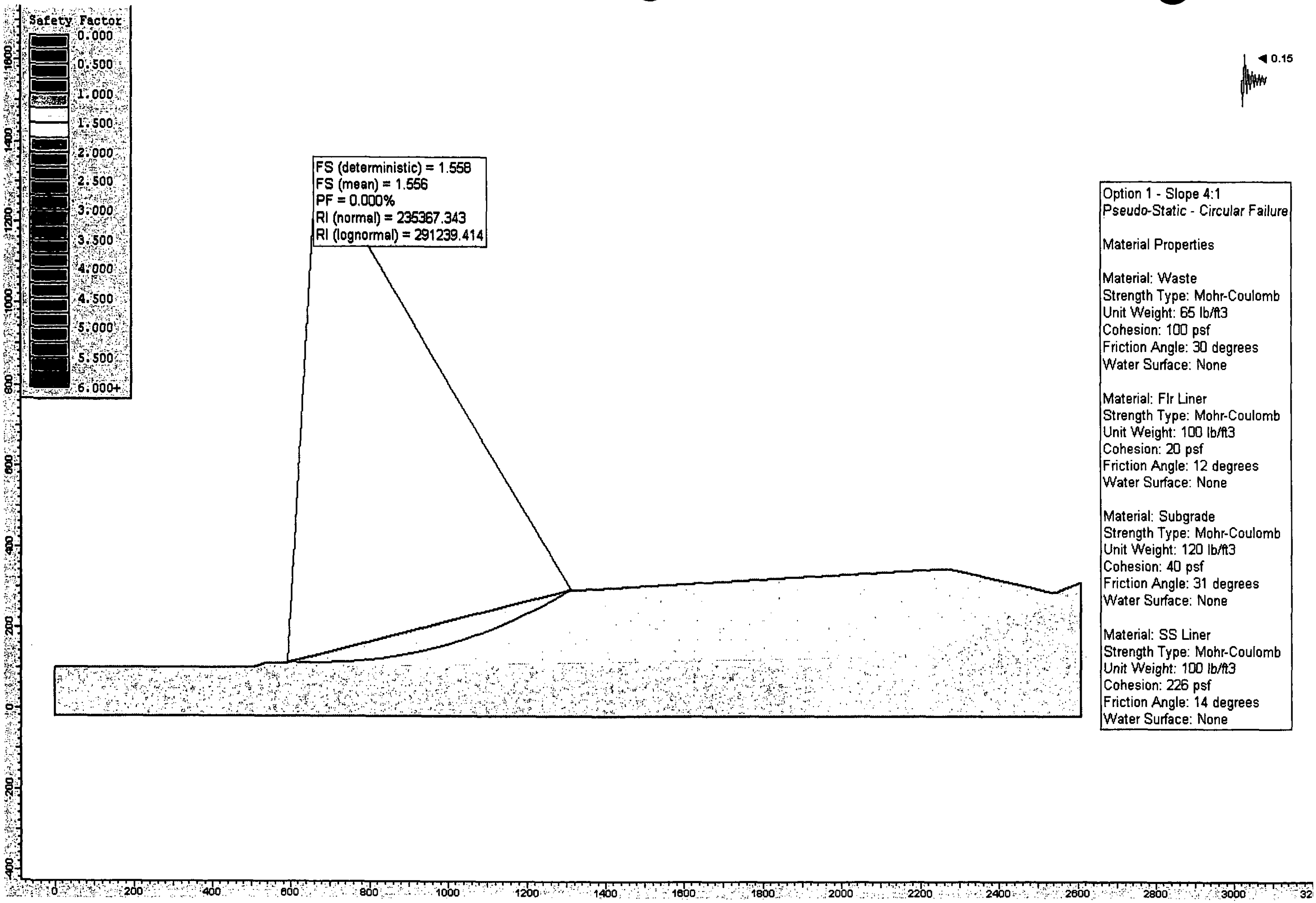
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 2.576
FS (mean) = 2.573
PF = 0.000%
Option 1 - Slope 4:1
Static - Circular Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None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0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 1.558
FS (mean) = 1.556
PF = 0.000%
RI (normal) = 235367.343
RI (lognormal) = 291239.414
0.15
Option 1 - Slope 4:1
Pseudo-Static - Circular Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
0
200
400
600
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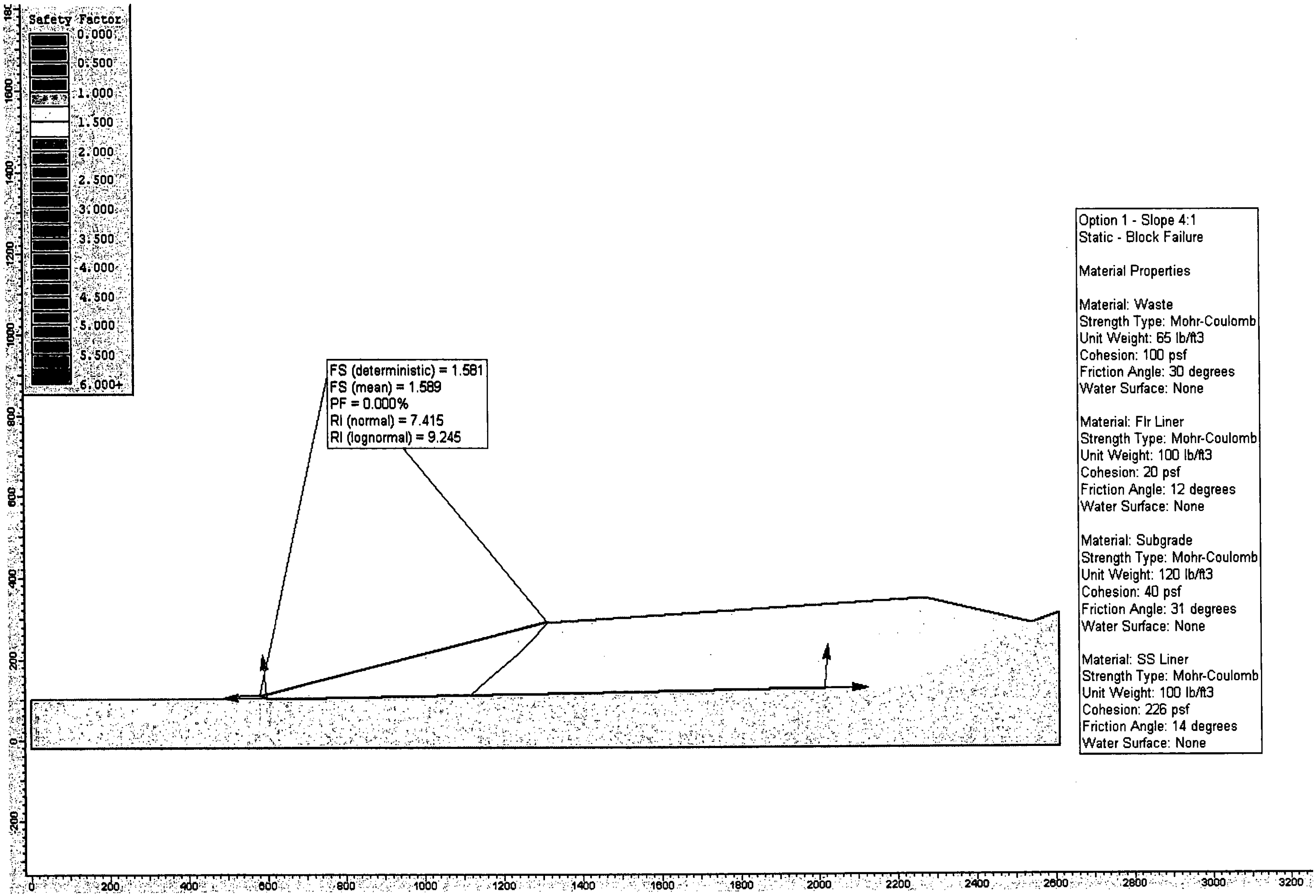
2400
2600
2800
3000



Safety Factor
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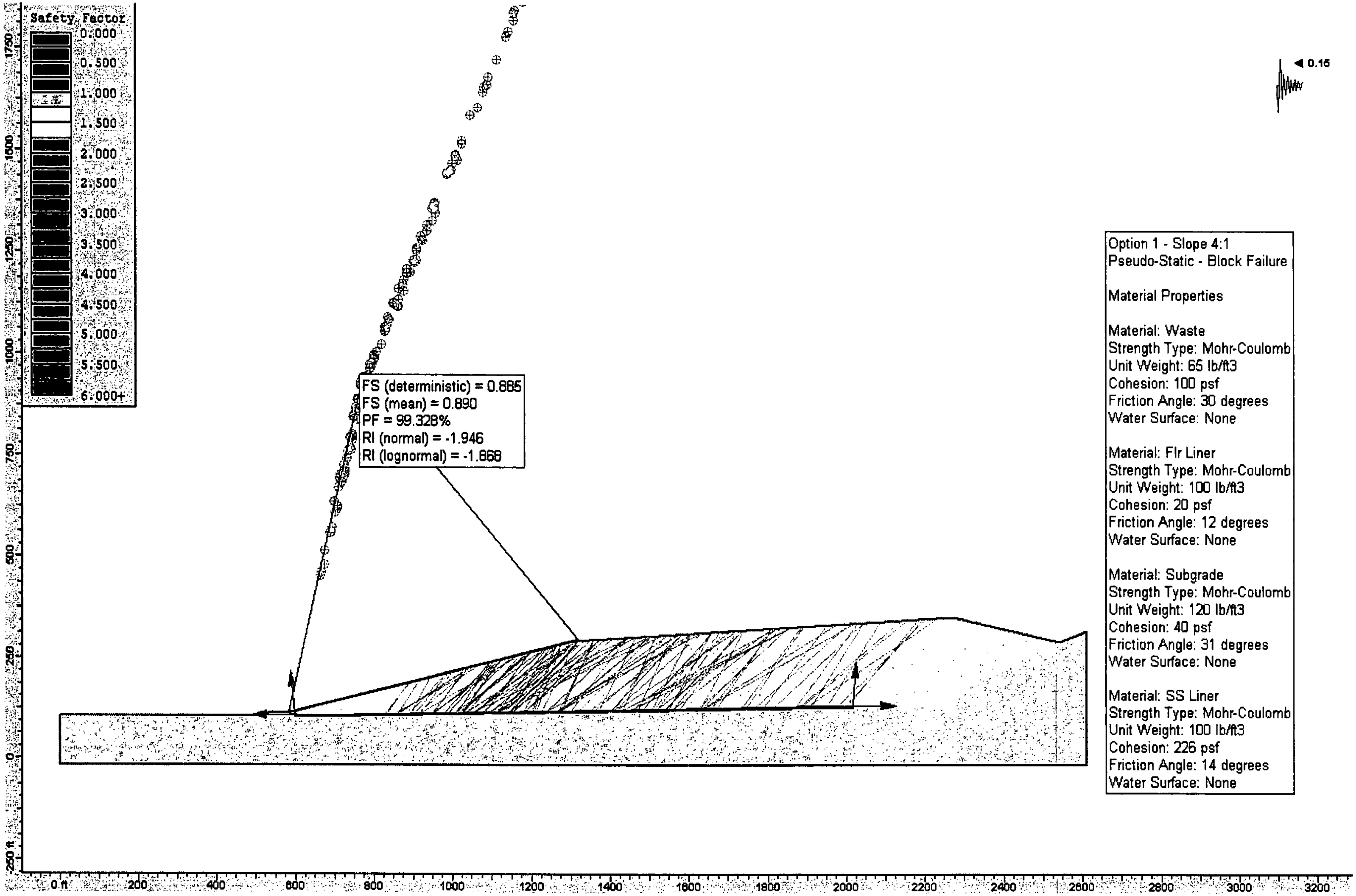
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 1.581
FS (mean) = 1.589
PF = 0.000%
RI (normal) = 7.415
RI (lognormal) = 9.245
Option 1 - Slope 4:1
Static - Block Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None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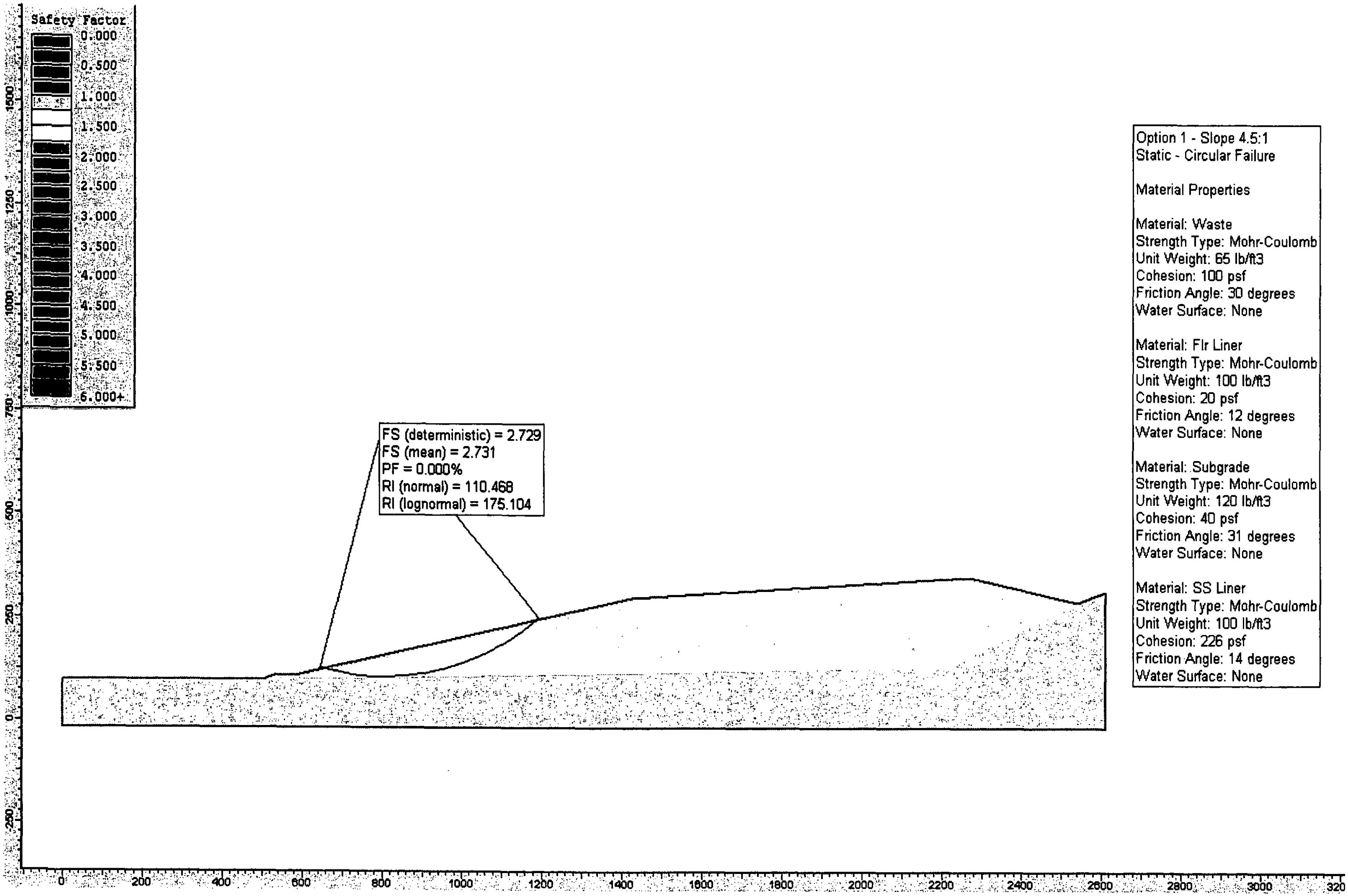
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.15
FS (deterministic) = 0.885
FS (mean) = 0.890
PF = 99.328%
RI (normal) = -1.946
RI (lognormal) = -1.868
Option 1 - Slope 4:1
Pseudo-Static - Block Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
1750
1500
1250
1000
750
500
250
0
-250 ft
0 ft
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200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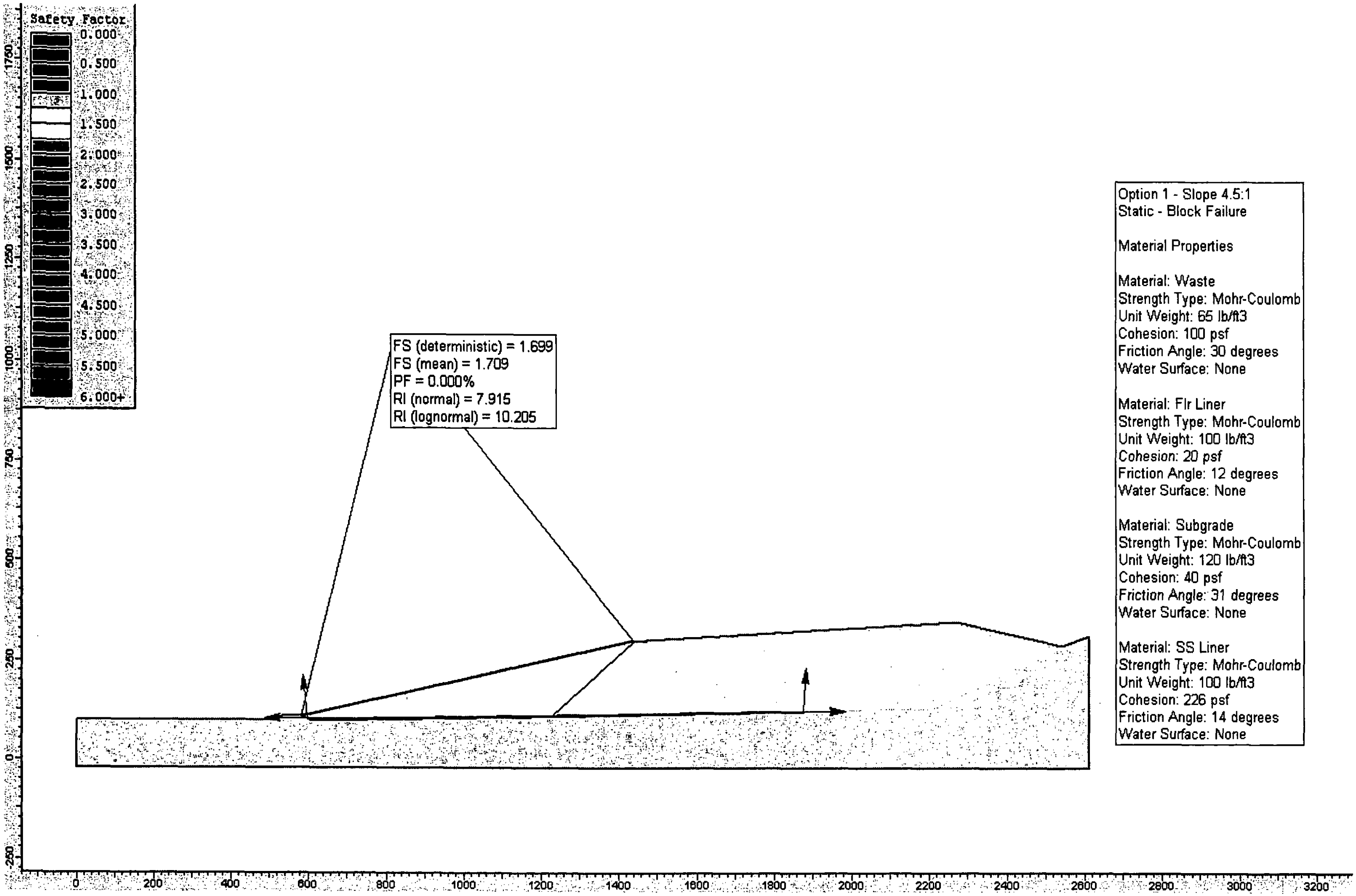
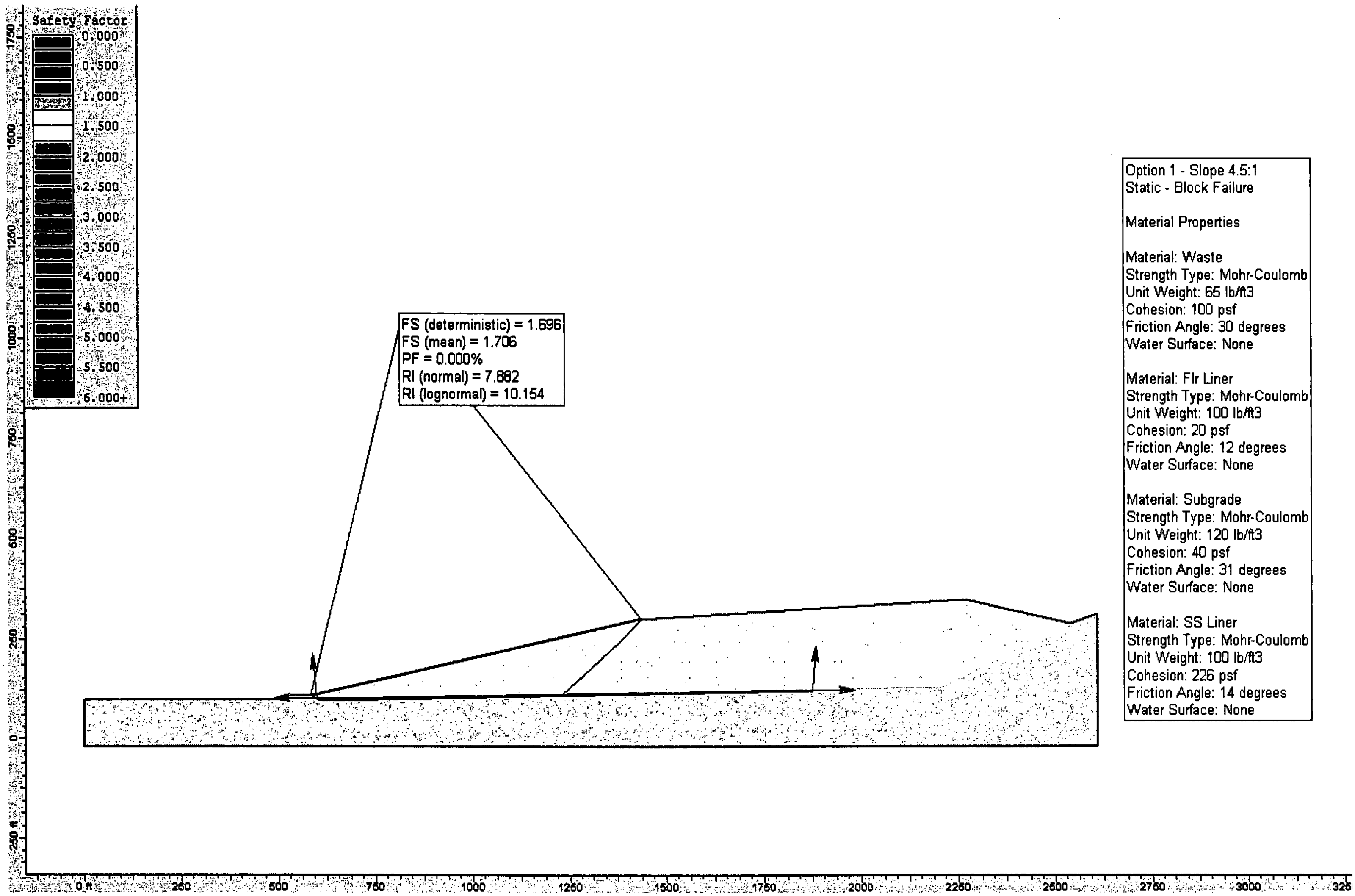
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 2.729
FS (mean) = 2.731
PF = 0.000%
RI (normal) = 110.468
RI (lognormal) = 175.104
Option 1 - Slope 4.5:1
Static - Circular Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None



Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 1.699
FS (mean) = 1.709
PF = 0.000%
RI (normal) = 7.915
RI (lognormal) = 10.205
Option 1 - Slope 4.5:1
Static - Block Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None



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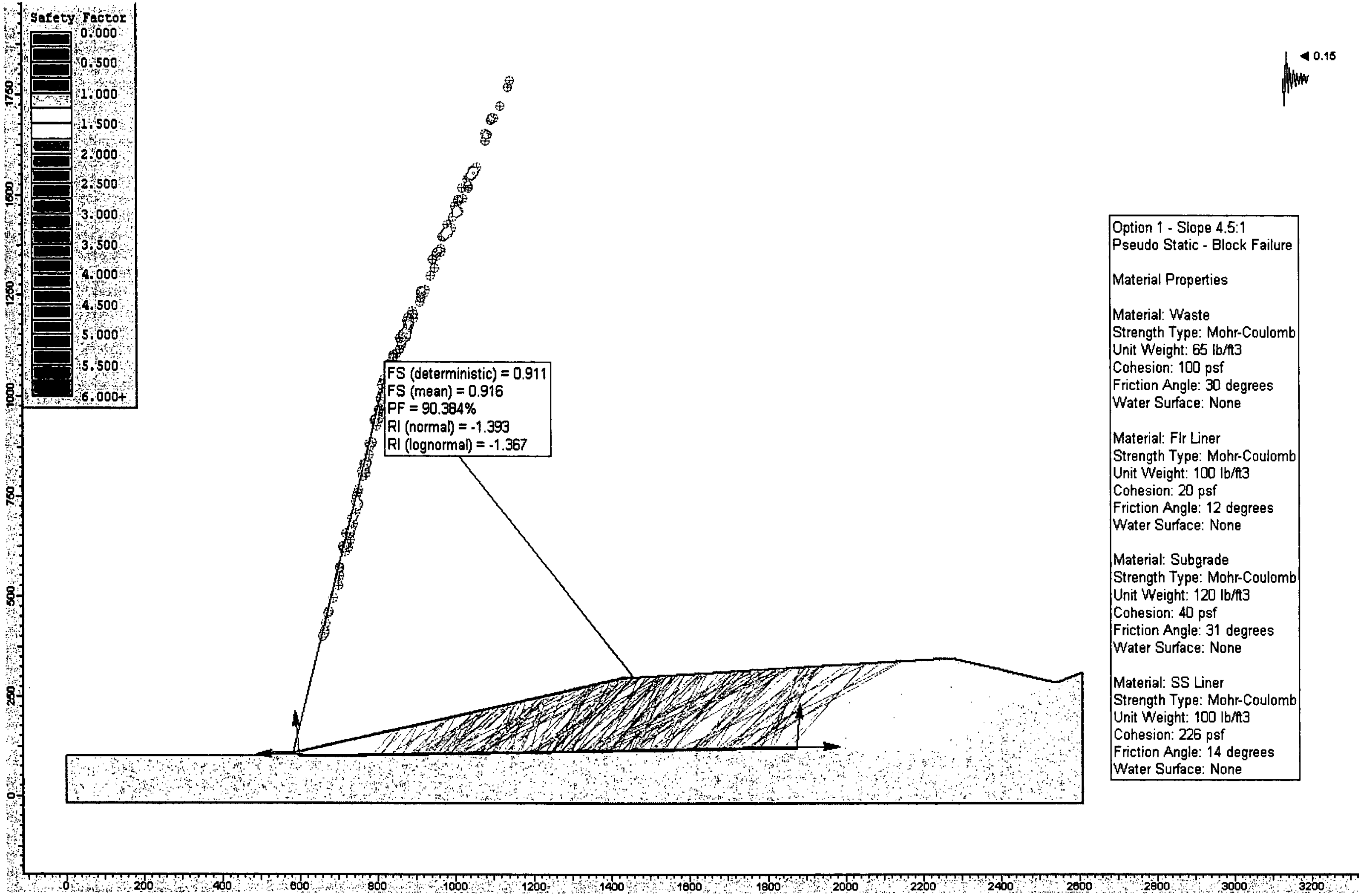
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 1.696
FS (mean) = 1.706
PF = 0.000%
RI (normal) = 7.882
RI (lognormal) = 10.154
Option 1 - Slope 4.5:1
Static - Block Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None



Safety Factor
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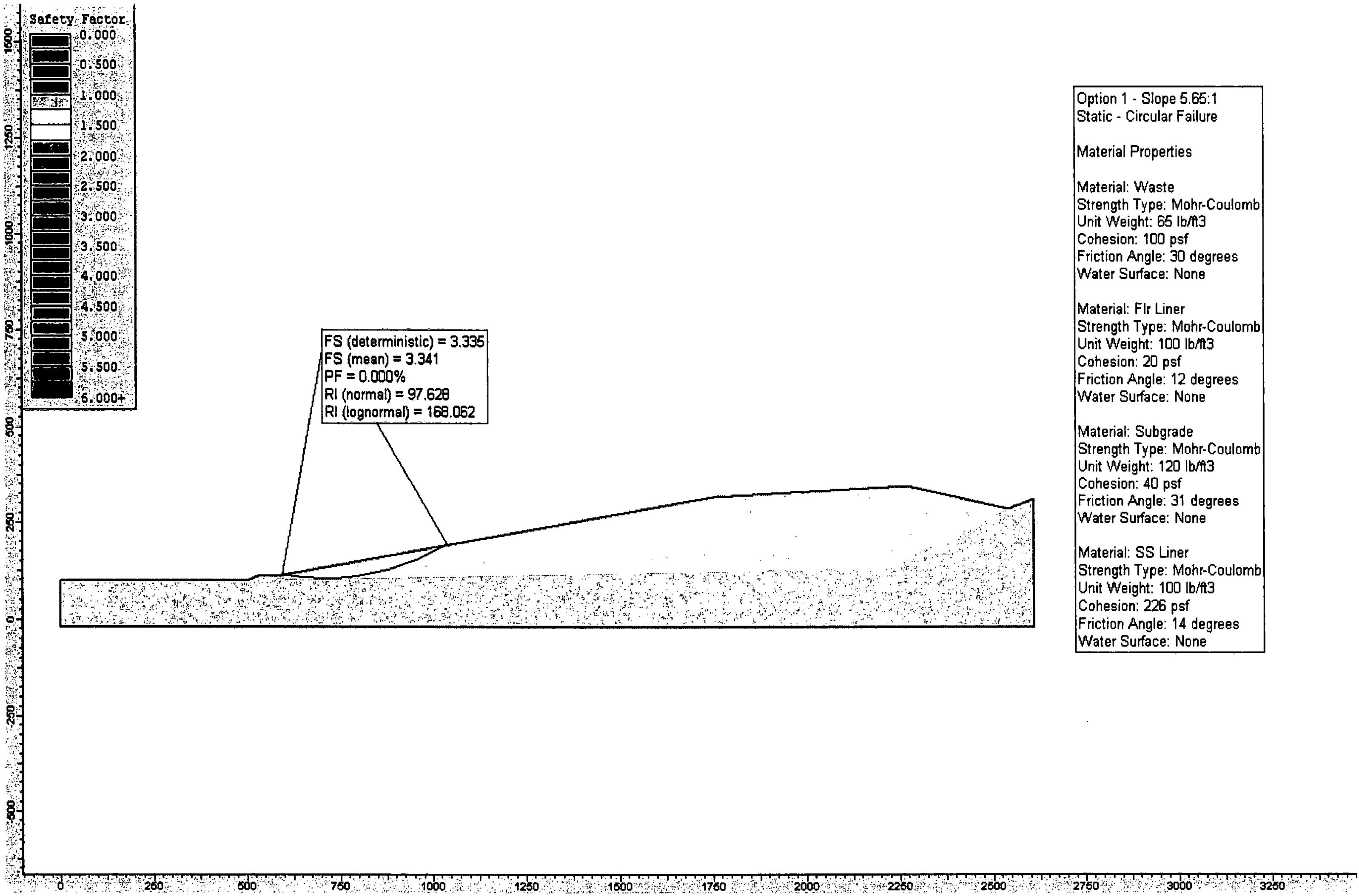
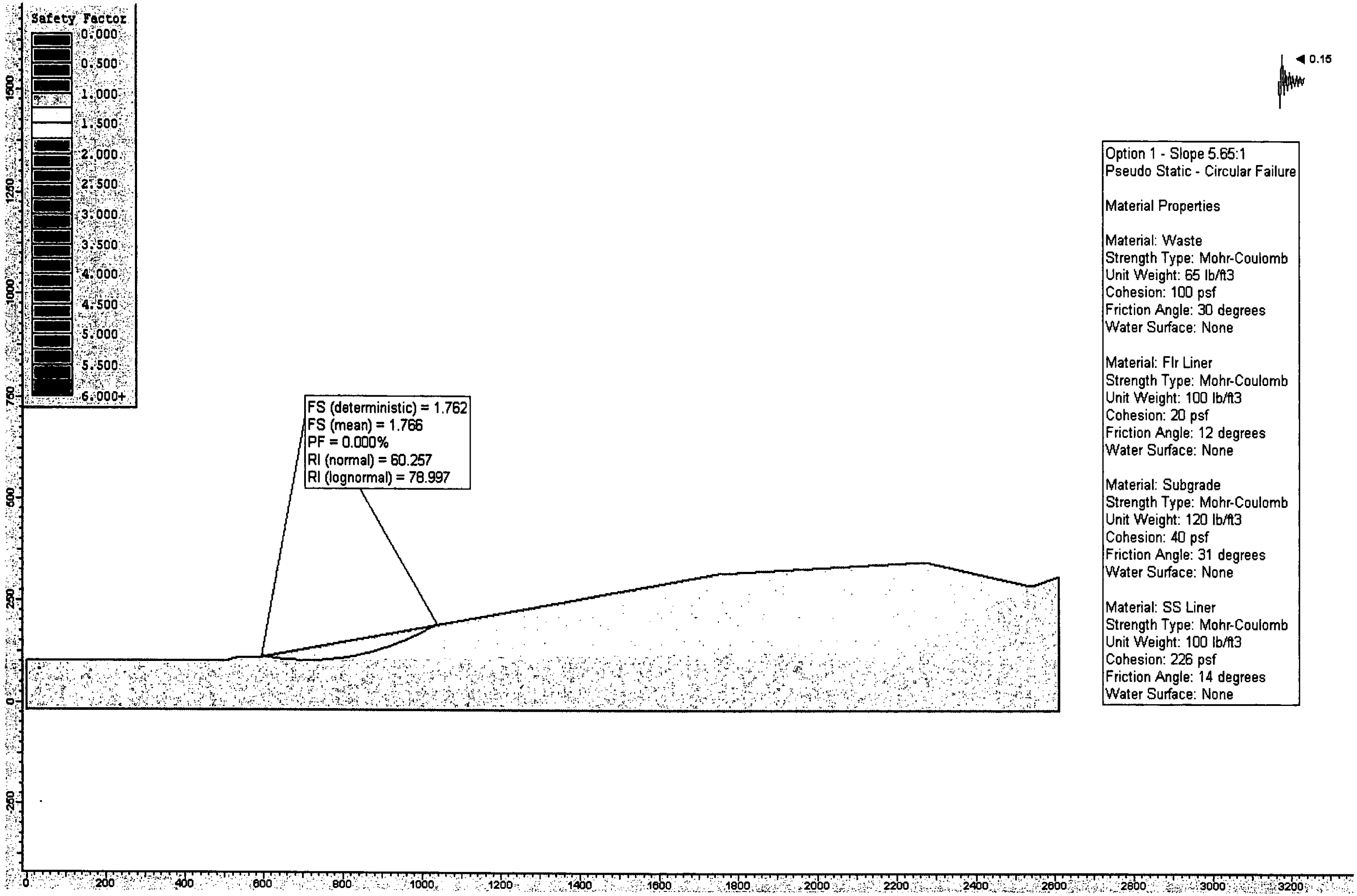
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.15
FS (deterministic) = 0.911
FS (mean) = 0.916
PF = 90.384%
RI (normal) = -1.393
RI (lognormal) = -1.367
Option 1 - Slope 4.5:1
Pseudo Static - Block Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None



Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 3.335
FS (mean) = 3.341
PF = 0.000%
RI (normal) = 97.628
RI (lognormal) = 168.062
Option 1 - Slope 5.65:1
Static - Circular Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None



Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.15
Option 1 - Slope 5.65:1
Pseudo Static - Circular Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
FS (deterministic) = 1.762
FS (mean) = 1.766
PF = 0.000%
RI (normal) = 60.257
RI (lognormal) = 78.997
1500
1250
1000
750
500
250
0
-250
0
200
400
600
800
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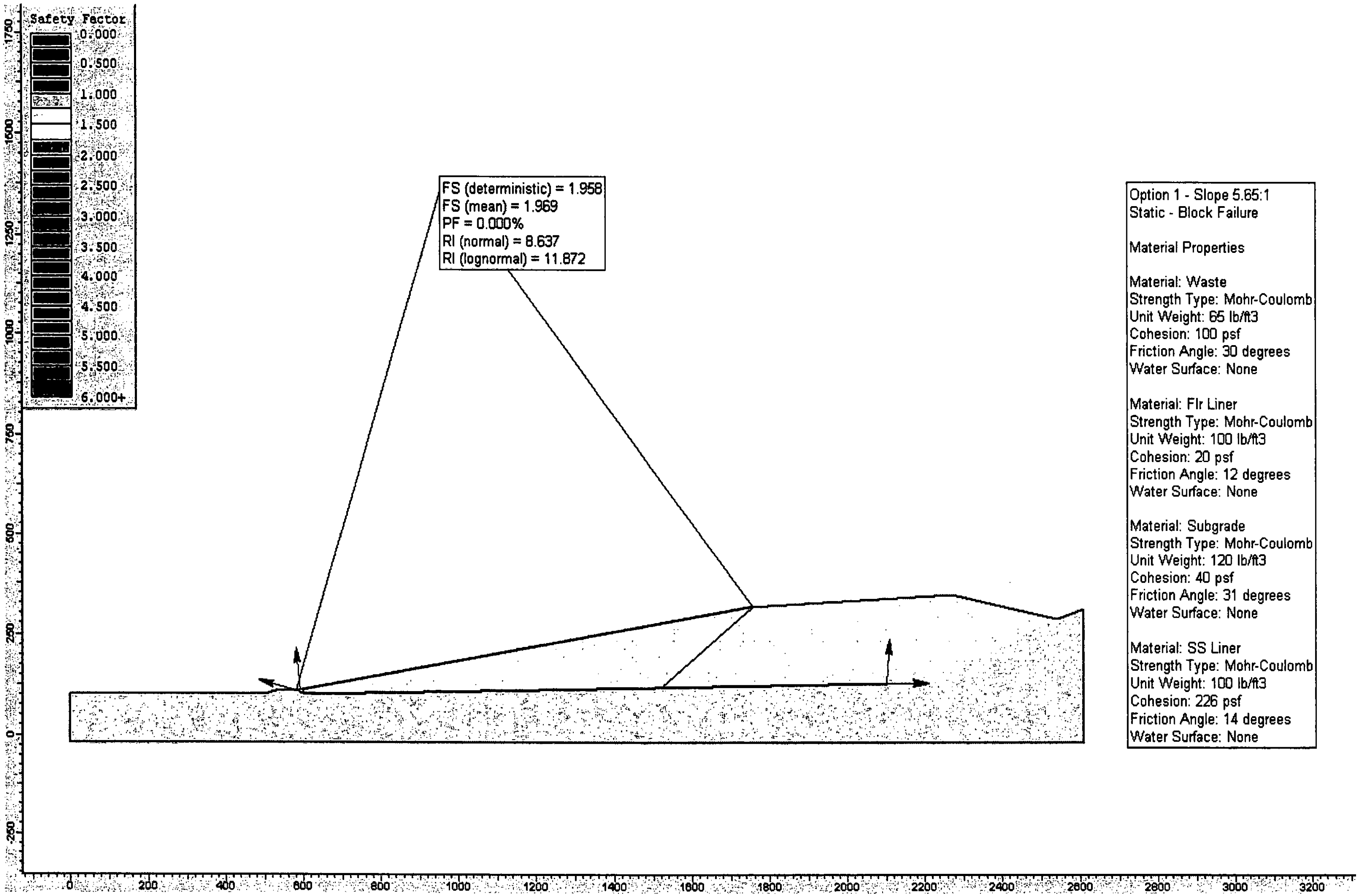
2600
2800
3000
3200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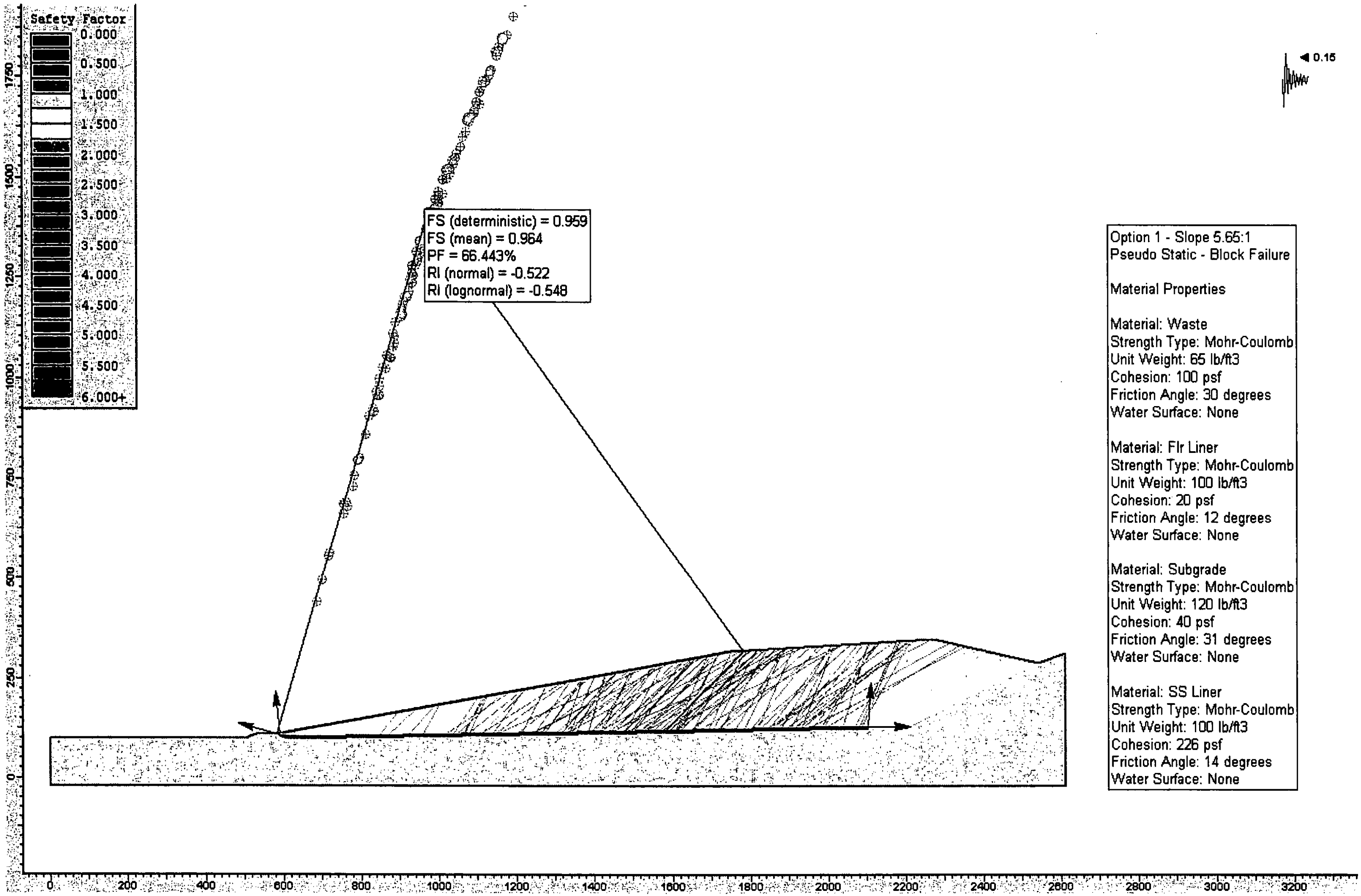
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 1.958
FS (mean) = 1.969
PF = 0.000%
RI (normal) = 8.637
RI (lognormal) = 11.872
Option 1 - Slope 5.65:1
Static - Block Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
1750
1500
1250
1000
750
500
250
0
-250
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200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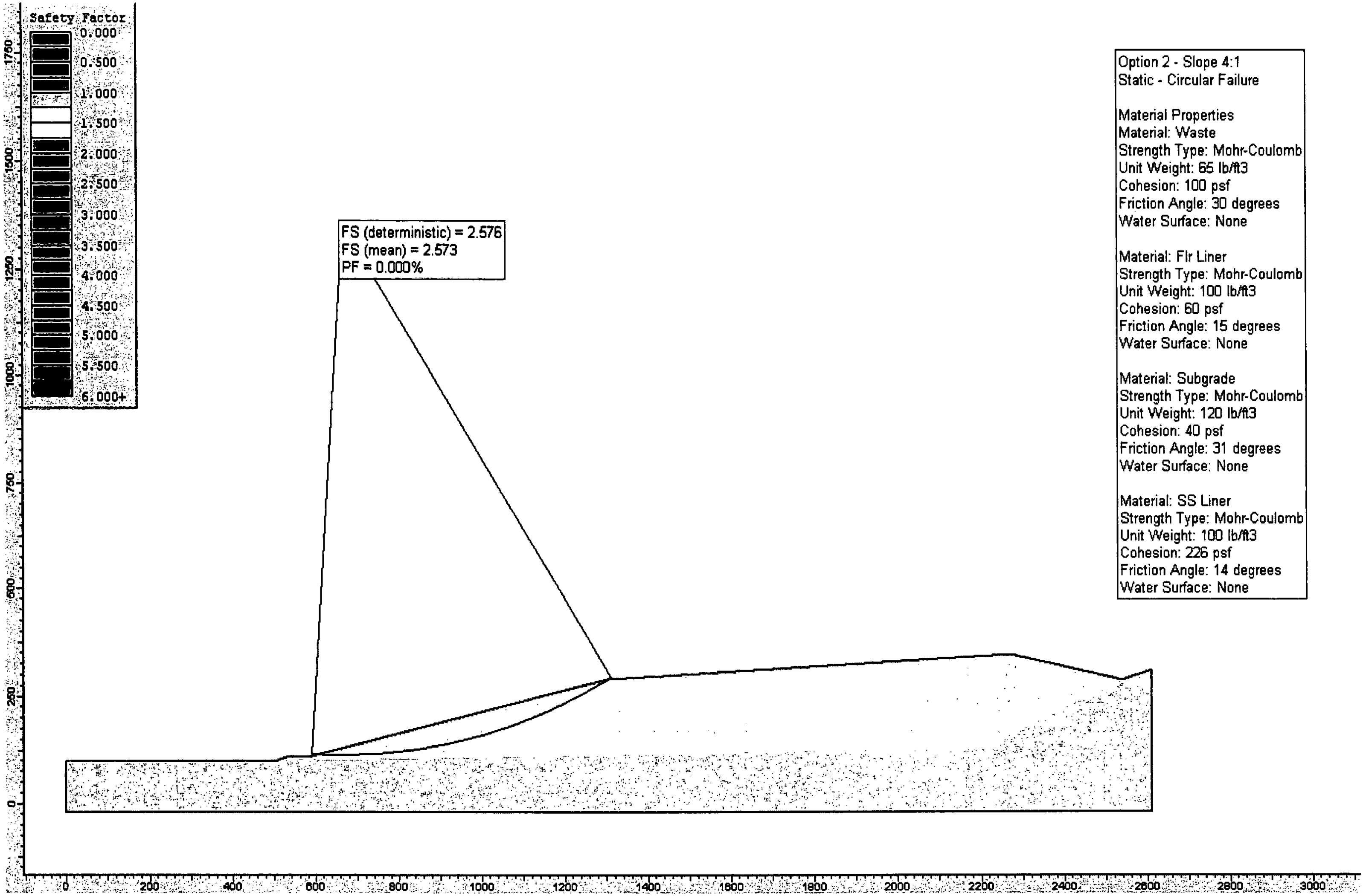
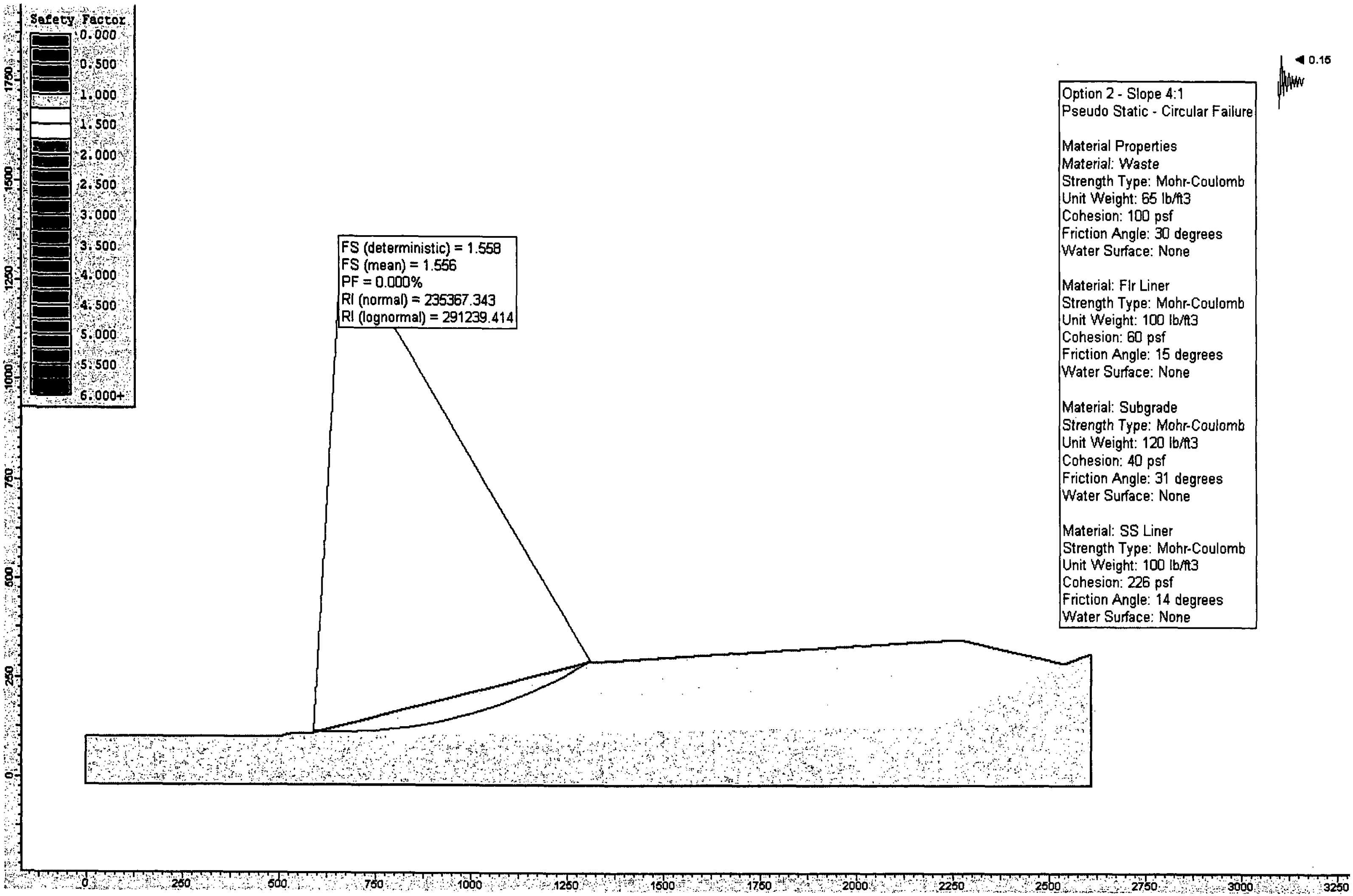
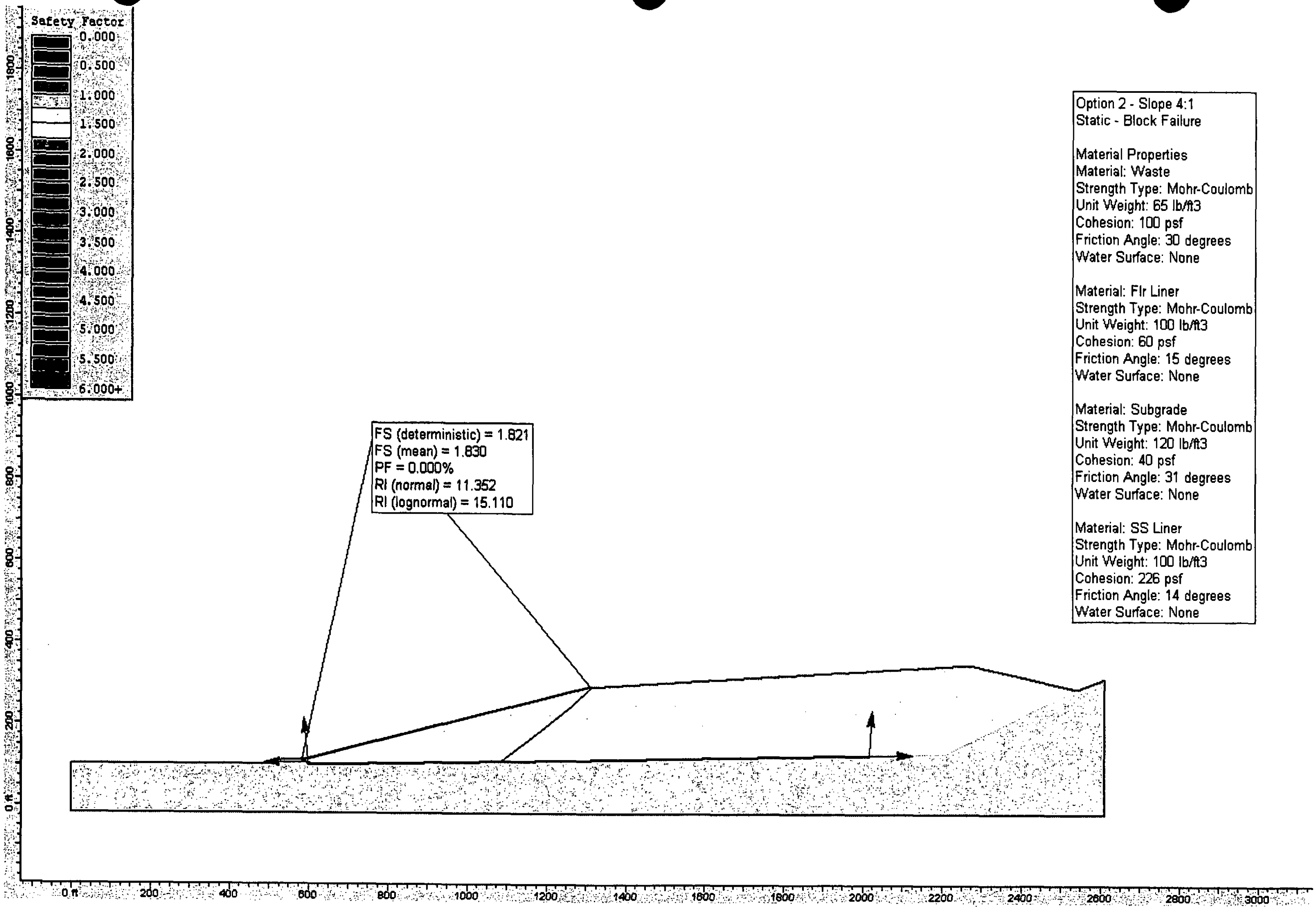
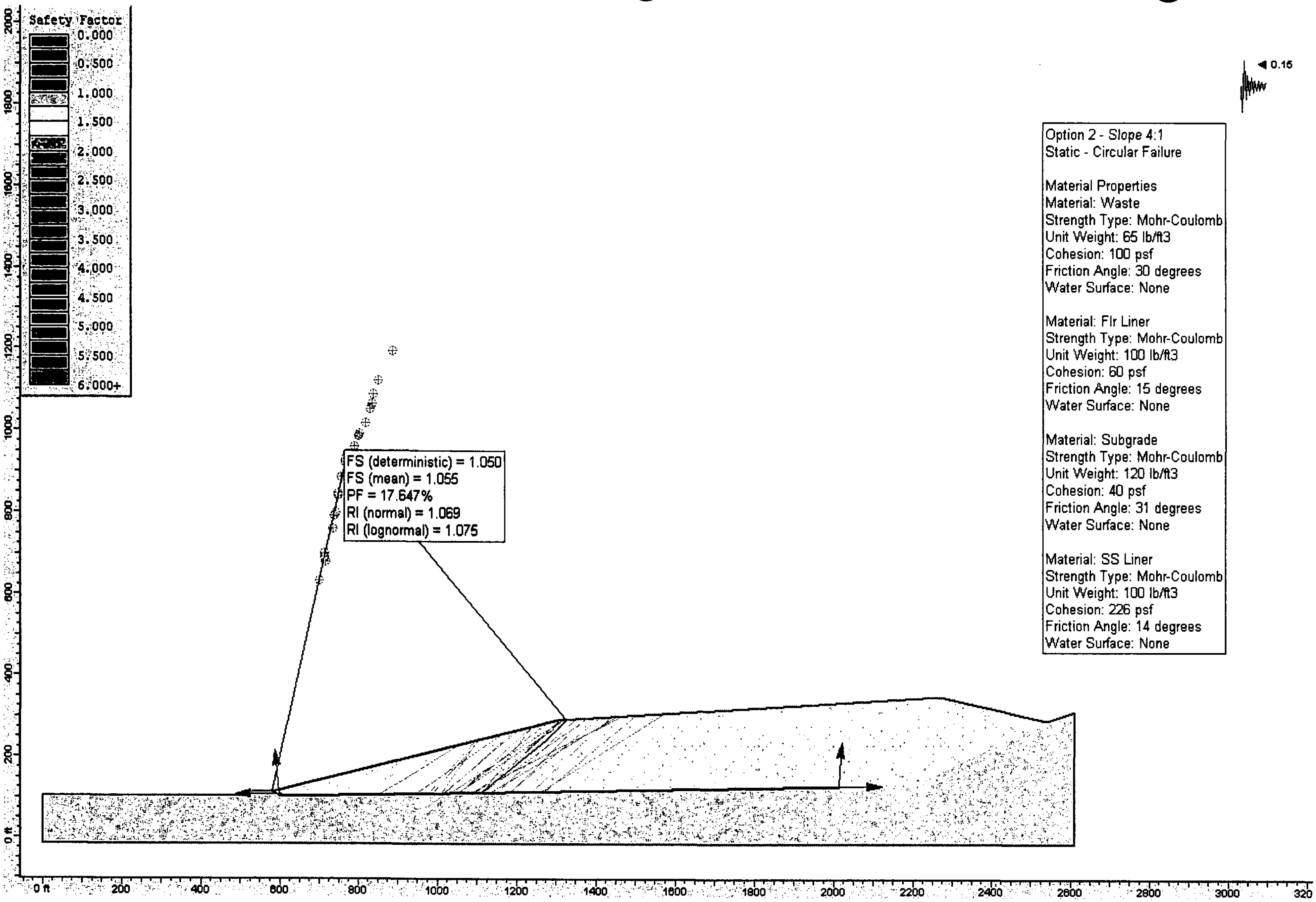
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 0.959
FS (mean) = 0.964
PF = 66.443%
RI (normal) = -0.522
RI (lognormal) = -0.548
0.15
Option 1 - Slope 5.65:1
Pseudo Static - Block Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 20 psf
Friction Angle: 12 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
0
250
500
750
1000
1250
1500
1750
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200



Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
FS (deterministic) = 2.576
FS (mean) = 2.573
PF = 0.000%
Option 2 - Slope 4:1
Static - Circular Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 60 psf
Friction Angle: 15 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.15
Option 2 - Slope 4:1
Pseudo Static - Circular Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 60 psf
Friction Angle: 15 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
FS (deterministic) = 1.558
FS (mean) = 1.556
PF = 0.000%
RI (normal) = 235367.343
RI (lognormal) = 291239.414
1750
1500
1250
1000
750
500
250
0
0
250
500
750
1000
1250
1500
1750
2000
2250
2500
2750
3000
3250


Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Option 2 - Slope 4:1
Static - Block Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 60 psf
Friction Angle: 15 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
FS (deterministic) = 1.821
FS (mean) = 1.830
PF = 0.000%
RI (normal) = 11.352
RI (lognormal) = 15.110
0 ft
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000



Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
0.15
Option 2 - Slope 4:1
Static - Circular Failure
Material Properties
Material: Waste
Strength Type: Mohr-Coulomb
Unit Weight: 65 lb/ft3
Cohesion: 100 psf
Friction Angle: 30 degrees
Water Surface: None
Material: Flr Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 60 psf
Friction Angle: 15 degrees
Water Surface: None
Material: Subgrade
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 40 psf
Friction Angle: 31 degrees
Water Surface: None
Material: SS Liner
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 226 psf
Friction Angle: 14 degrees
Water Surface: None
FS (deterministic) = 1.050
FS (mean) = 1.055
PF = 17.647%
RI (normal) = 1.069
RI (lognormal) = 1.075